Standards & Guidelines for

Municipal Waterworks, Wastewater & Storm Drainage Systems

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January 2006

TABLE OF CONTENTS

FOREWORD ... xi
ACKNOWLEDGEMENTS ... xiii
DEFINITIONS / ABBREVIATIONS ... xv

1.0 Waterworks Systems Standards ... 1-1

1.1 Potable Water Quality Standards ... 1-1

1.2 Potable Water Treatment Standards ... 1-1
1.2.1 Surface Water and GWUDI ... 1-1
1.2.1.1 Filtration and Disinfection Requirement ... 1-1
1.2.1.2 Filtration Exemption ... 1-2
1.2.1.3 Source Water Monitoring Requirements for Filtered Systems ... 1-2
1.2.1.4 Criteria to Determine GWUDI ... 1-3
1.2.2 Groundwater ... 1-3

1.3 Potable Water Treatment Performance Standards ... 1-4
1.3.1 Filtration ... 1-4
1.3.1.1 Rapid Sand Filtration ... 1-4
1.3.1.2 Slow Sand Filtration ... 1-4
1.3.1.3 Membrane filtration ... 1-5
1.3.1.4 Cartridge Filtration ... 1-5
1.3.2 Disinfection ... 1-6
1.3.2.1 Chlorine (Free and Combined) ... 1-6
1.3.2.2 Ultraviolet Light ... 1-7
1.3.2.3 Chlorine dioxide ... 1-8
1.3.2.4 Ozone ... 1-8
1.3.3 Fluoridation ... 1-9

1.4 Potable Water Treatment Credits ... 1-9
1.4.1 Filtration Credit ... 1-9
1.4.1.1 System Non-compliance with respect to turbidity reduction ... 1-9
1.4.2 Disinfection Credit ... 1-10
1.4.3 Other Options Credit ... 1-10

1.5 Potable Water Treatment Facility Design Standards ... 1-11
1.5.1 Surface Water and GWUDI ... 1-11
1.5.1.1 Rapid Sand Filtration ... 1-11
1.5.1.2 Slow Sand Filtration ... 1-13
1.5.1.3 Membrane Filtration ... 1-14
1.5.1.4 Cartridge Filtration ... 1-20
1.5.2 Groundwater ... 1-21
1.5.3 Disinfection ... 1-21
1.5.3.1 Chlorine / Chloramines ... 1-21
1.5.3.2 Ultraviolet Light ... 1-29

1.5.3.3 Chlorine dioxide 1-32
1.5.3.4 Ozone 1-32
1.5.4 Fluoridation 1-33
1.5.4.1 General Requirements 1-33
1.5.4.2 Chemical Feed 1-34
1.5.4.3 Metering 1-34
1.5.4.4 Alternate Compounds 1-35
1.5.4.5 Chemical Storage and Ventilation 1-35
1.5.4.6 Record of Performance Monitoring 1-35
1.5.4.7 Sampling 1-35
1.5.4.8 Safety 1-36
1.5.4.9 Repair and Maintenance 1-36
1.5.5 Health and Safety Act 1-36

1.6 Waterworks Systems Operation Standards 1-37
1.6.1 System Operation Program 1-37
1.6.2 Reliability 1-37
1.6.3 Operations 1-37

1.7 Potable Water Treatment Chemical standards 1-40
1.7.1 Use, Storage and Handling 1-40
1.7.2 Direct and Indirect Additives 1-40
1.7.2.1 Indirect Additives 1-40
1.7.2.2 Direct Additives 1-40

1.8 Potable Water Treatment Plant Waste – Handling and Disposal Standards 1-41
1.8.1 Sanitary Wastes 1-41
1.8.2 Filter Backwash 1-41
1.8.3 Filter-to-Waste 1-41
1.8.4 Sludges 1-41
1.8.4.1 Aluminum Sludge 1-41
1.8.4.2 Lime Sludge 1-42
1.8.4.3 Sludges from Other Coagulants 1-42

1.9 Potable Water Transmission and Distribution Main Standards 1-43
1.9.1 Pipe Performance Standards 1-43
1.9.2 Frost Protection for Mains and Reservoirs 1-43
1.9.2.1 Mains 1-43
1.9.2.2 Reservoirs 1-43
1.9.3 Cross-Connection Controls 1-43
1.9.3.1 Horizontal Separation of Water Mains and Sewers 1-43
1.9.3.2 Pipe Crossings 1-44
1.9.3.3 Valve, Air Relief, Meter and Blow-Off Chambers 1-44
1.9.3.4 Backflow Prevention and Control 1-44
1.9.4 Disinfection of Mains and Reservoirs 1-44
1.9.4.1 Mains 1-44
1.9.4.2 Reservoirs 1-45
1.9.4.3 Discharge of Superchlorinated Water 1-45
1.9.5 Layout 1-45

1.10 Potable Water Quality Performance Monitoring Standards 1-45
1.10.1 General 1-45

1.10.2 Analytical Requirements....1-46
1.10.3 Compliance Monitoring....1-46
1.10.3.1 Bacteriological....1-46
1.10.3.2 Physical Parameters, Organic & Inorganic Chemicals and Pesticides....1-47
1.10.3.3 Turbidity....1-49
1.10.3.4 Fluoride....1-49
1.10.3.5 Iron and Manganese....1-50
1.10.3.6 Trihalomethanes and Bromodichloromethane....1-50
1.10.3.7 Disinfection....1-53
1.10.4 Issue Oriented and Follow-up Monitoring....1-57
1.10.4.1 General....1-57
1.10.4.2 Bacteriological....1-57
1.10.4.3 Disinfection....1-57
1.10.4.4 Fluoride....1-58
1.10.4.5 Turbidity....1-58
1.10.4.6 Organic and Inorganic Chemicals, Pesticides, THM and BDCM....1-58

1.11 Potable Water Quality Record Keeping and Reporting Standards....1-58
1.11.1 Record Keeping....1-58
1.11.1.1 Five Year Records....1-58
1.11.1.2 Lifetime Records....1-59
1.11.1.3 Sampling and Analysis Records....1-60
1.11.2 Reporting Requirements....1-60
1.11.2.1 Contravention Reporting....1-60
1.11.2.2 Monthly Reporting....1-61
1.11.2.3 Annual Reporting....1-62
1.11.2.4 Electronic Reporting....1-62

1.12 Laboratory Data Quality Assurance Standards....1-62

1.13 Facility Risk Assessment Standards....1-63

1.14 Facility Classification and Operator Certification Standards....1-63

2.0 Waterworks Systems Guidelines....2-1

2.1 Design Criteria....2-1
2.1.1 Water Quantity Requirements....2-1
2.1.2 System Capacity....2-1
2.1.3 Raw Water Supply and Water Treatment....2-1
2.1.4 Water Distribution....2-1

2.2 Raw Water Supply....2-2
2.2.1 Source Water Protection....2-2
2.2.1.1 Surface Water or Groundwater under Direct Influence of Surface Water Sources....2-3
2.2.1.2 Groundwater....2-4
2.2.1.3 Summary....2-5
2.2.2 Water Source/Quality....2-5
2.2.3 Site Selection Criteria....2-5
2.2.4 Surface Water Supply....2-6
2.2.4.1 Intakes....2-6
2.2.4.2 Screening....2-7

2.2.4.3 Pumping 2-7
2.2.4.4 Raw Water Storage 2-8
2.2.5 Groundwater Supply 2-10
2.2.5.1 Siting of Wells 2-10
2.2.5.2 Well Protection 2-10
2.2.5.3 Pumphouse Design 2-10
2.2.5.4 Well Disinfection 2-11

2.3 Water Treatment 2-11
2.3.1 Rapid Sand Filtration 2-11
2.3.1.1 General Guidelines 2-11
2.3.1.2 Chemical Mixing and Coagulation 2-13
2.3.1.3 Flocculation 2-15
2.3.1.4 Solids Separation 2-17
2.3.1.5 Filtration 2-21
2.3.1.6 Flow Measurements 2-25
2.3.2 Slow Sand Filtration 2-26
2.3.2.1 Requisite Conditions 2-26
2.3.2.2 Filter Media 2-27
2.3.2.3 Filter Depth 2-27
2.3.2.4 Filtration Rates 2-27
2.3.2.5 Headloss 2-28
2.3.2.6 Hydraulics 2-28
2.3.2.7 Gravel Support 2-30
2.3.2.8 Flow Measurements 2-30
2.3.4 Lead and Copper 2-31
2.3.5 Iron and Manganese Control 2-31
2.3.5.1 Removal of Iron and Manganese 2-31
2.3.5.2 Control of Iron and Manganese 2-32
2.3.6 Controls and Instrumentation 2-33
2.3.6.1 General 2-33
2.3.6.2 Measurement List 2-33
2.3.6.3 Degree of Automation in Plant Control 2-35
2.3.6.4 Alarms and Status Indication 2-36
2.3.6.5 Control Equipment (Automatic Systems) 2-37
2.3.6.6 Network Security 2-38
2.3.6.7 Historical Data 2-38
2.3.7 Colour Codes for Water Treatment Plant Piping 2-39
2.3.8 Laboratory Requirements 2-39
2.3.9 Operational Monitoring 2-40
2.3.10 Water Treatment Chemicals 2-42
2.3.10.1 Labels and Material Safety Data Sheets 2-42
2.3.10.2 Storage and Handling 2-42

2.4 Water Transmission and Distribution Mains 2-44
2.4.1 Pipe Sizing 2-44
2.4.2 Valves and Hydrants 2-45
2.4.2.1 Valve Placement 2-45
2.4.2.2 Fire Flows and Hydrants 2-45
2.4.3 Pumping 2-45

2.5 Potable Water Storage 2-46
2.5.1 Sizing 2-46
2.5.2 Phasing 2-46
2.5.3 Alternative Types 2-46
2.5.4 Site Selection 2-46
2.5.5 Design Considerations 2-47
2.5.5.1 Multi-Cell Provision 2-47
2.5.5.2 Access 2-47
2.5.5.3 Vents 2-47
2.5.5.4 Drains and Overflows 2-47
2.5.5.5 Circulation 2-47

2.6 Watering Point 2-48

2.7 Water Systems Security and Protection 2-48
2.7.1 Vulnerability Assessment 2-48
2.7.2 Emergency Reaction Plans 2-48
2.7.3. Physical Security 2-49
2.7.3.1 Personnel 2-49
2.7.3.2 Training 2-49
2.7.3.3 Access Control 2-50
2.7.3.4 Perimeter Security 2-50
2.7.3.5 Detection Systems 2-50
2.7.4 SCADA Security 2-51
2.7.4.1 Critical Cyber Assets 2-51
2.7.4.2 Identification of Connections 2-51
2.7.4.3 Authentication and Access 2-51
2.7.4.4 Anti-Virus 2-51
2.7.4.5 Operating Systems 2-51
2.7.4.6 Network Logging 2-52
2.7.5 Excavations and Open Trenches 2-52
2.7.6 Reservoirs 2-52

2.8 Facility Risk Assessment 2-52

2.9 Conservation and Reclamation 2-56

3.0 Wastewater Systems - Performance Standards 3-1

3.1 Treated Effluent Disposal to Surface Waters 3-1
3.1.1 Treated Effluent Quality 3-1
3.1.2 Best Practicable Technology Standards 3-1
3.1.3 Receiving Water Quality Based Standards 3-1
3.1.4 Disposal Criteria 3-1

3.2 Treated Effluent Disposal to Land 3-4
3.2.1 Wastewater Irrigation 3-4
3.2.1.1 Minimum Treatment Requirement 3-4
3.2.1.2 Treated Effluent Quality Standards 3-4
3.2.2 Rapid Infiltration 3-5
3.2.2.1 Minimum Treatment Requirement 3-5
3.2.3 Wetlands Disposal 3-5

3.2.3.1 Minimum Treatment Requirement 3-5

4.0 Wastewater Systems - Design Standards 4-1

4.1 Wastewater Collection 4-1
4.1.1 Sanitary Sewers 4-1
4.1.1.1 Minimum Pipe Diameter 4-1
4.1.1.2 Minimum Pipe Slope 4-1
4.1.1.3 Minimum Flow Depths 4-1
4.1.1.4 Solids Deposition 4-2
4.1.1.5 Alignment 4-2
4.1.1.6 Frost Protection 4-2
4.1.1.7 Cross Connections 4-2
4.1.2 Wastewater Pump Stations 4-3
4.1.2.1 Site Constraints 4-3
4.1.2.2 Pumps 4-3
4.1.2.3 Separation 4-4
4.1.2.4 Access 4-4
4.1.2.5 Water Supply 4-5
4.1.3 Screens and Grit Removal Facilities 4-5
4.1.3.1 Access and Ventilation 4-5
4.1.4 Health and Safety Act 4-5

4.2 Wastewater Treatment and Disposal 4-6
4.2.1 Wastewater Lagoons 4-6
4.2.1.1 General Requirements 4-6
4.2.1.2 System Components and Configuration 4-6
4.2.1.3 Design Considerations 4-9
4.2.1.4 Seepage Control 4-10
4.2.1.5 Construction Features 4-13
4.2.2 Wastewater Evaporation Lagoons 4-14
4.2.2.1 System Components and Configuration 4-14
4.2.2.2 Design Considerations 4-15
4.2.3 Mechanical Wastewater Treatment 4-15
4.2.3.1 General Requirements 4-15
4.2.3.2 Design Considerations 4-15
4.2.3.3 Safety 4-15
4.2.3.4 Water Supply and Sanitary Facilities 4-17
4.2.4 Aerated Lagoons 4-18
4.2.4.1 General Requirements 4-18
4.2.4.2 System Components and Configuration 4-18
4.2.4.3 Design Considerations 4-18
4.2.5 Disinfection 4-19
4.2.5.1 General Requirements 4-19
4.2.6 Wastewater Treatment Chemicals 4-19
4.2.7 Effluent Disposal 4-19
4.2.7.1 Disposal to Surface Waters 4-19
4.2.7.2 Disposal to Land 4-20
4.2.8 Biosolids (Sludge) Disposal 4-21
4.2.9 Health and Safety Act 4-22

5.0 Wastewater Systems - Design Guidelines 5-1

5.1 Design Criteria 5-1
5.1.1 Estimating Wastewater Flows 5-1
5.1.1.1 Residential (Population-Generated) 5-1
5.1.1.2 Commercial/Institutional and Industrial 5-1
5.1.1.3 Extraneous Flow Allowance - All Land Uses 5-4
5.1.1.4 Total Peak Design Flow Rates 5-4
5.1.2 System Capacity 5-5
5.1.3 Wastewater Collection and Treatment System 5-5
5.1.4 Sewer Outfall 5-5

5.2 Wastewater Collection 5-6
5.2.1 Sewers 5-6
5.2.1.1 Materials 5-6
5.2.1.2 Sizing of Sewers 5-6
5.2.1.3 Changes in Pipe Size 5-7
5.2.1.4 Location 5-7
5.2.1.5 Pressure Testing 5-7
5.2.2 Manholes 5-7
5.2.2.1 Location 5-7
5.2.2.2 Sizing 5-8
5.2.2.3 Drop Manholes 5-8
5.2.2.4 Channelling and Benching 5-8
5.2.2.5 Frame and Cover 5-8
5.2.2.6 Steps 5-9
5.2.3 Inverted Siphons 5-9
5.2.4 Wastewater Pump Station 5-9
5.2.4.1 General 5-9
5.2.4.2 Wet Well/Dry Well Pump Station 5-10
5.2.4.3 Suction-Lift Pump Station 5-12
5.2.4.4 Submersible Pump Stations 5-13
5.2.4.5 Alarm Systems 5-14
5.2.4.6 Emergency Operation 5-14
5.2.5 Forcemains 5-17
5.2.5.1 Velocity and Diameter 5-17
5.2.5.2 Air and Vacuum Relief Valve 5-17
5.2.5.3 Termination 5-17
5.2.5.4 Design Pressure 5-17
5.2.6 Security of Open Trenches and Excavations 5-17

5.3 Wastewater Treatment 5-18
5.3.1 Mechanical Wastewater Treatment 5-18
5.3.1.1 Site Selection 5-18
5.3.1.2 Plant Hydraulics 5-19
5.3.1.3 Wastewater Characterization 5-20
5.3.1.4 Preliminary Treatment 5-21
5.3.1.5 Primary Treatment 5-25
5.3.1.6 Secondary Treatment 5-26
5.3.1.7 Tertiary Treatment 5-42
5.3.1.8 Secondary Clarifier 5-59
5.3.1.9 Laboratory Requirements 5-60

5.3.1.10 Flow Measurements 5-60
5.3.1.11 Colour Codes 5-62
5.3.2 Aerated Lagoons 5-63
5.3.2.1 General 5-63
5.3.2.2 Design Approach 5-63
5.3.2.3 System Design (Aeration) 5-65
5.3.3 Odour Control 5-67
5.3.3.1 Odour Production 5-67
5.3.3.2 Potential Odour Sources 5-68
5.3.3.3 Evaluation of Odour Production Potential 5-69
5.3.3.4 Odour Control and Abatement Measures 5-70
5.3.4 Instrumentation and Controls 5-73
5.3.4.1 General 5-73
5.3.4.2 Types of Instruments 5-74
5.3.4.3 Process Controls 5-77
5.3.4.4 Design Documents 5-82
5.3.4.5 Control System Documentation 5-82
5.3.4.6 Training 5-83
5.3.5 Emergency Facilities and Component Reliability 5-83
5.3.5.1 Back-Up Requirements 5-83
5.3.5.2 Wastewater Bypass Facilities 5-84
5.3.5.3 Standby Power 5-84

5.4 Residual/Biosolids (Sludge) Processing 5-85
5.4.1 General 5-85
5.4.2 Biosolids (Sludge) Handling and Treatment 5-85
5.4.2.1 Digestion 5-85
5.4.2.2 Conditioning 5-89
5.4.2.3 Thickening 5-90
5.4.2.4 Dewatering 5-93

6.0 Stormwater Management Guidelines 6-1

6.1 General 6-1

6.2 Stormwater Collection 6-1
6.2.1 Dual Drainage Concept 6-1
6.2.1.1 Design Capacity 6-1
6.2.2 Storm Sewers 6-1
6.2.2.1 Sewer Hydraulics 6-1
6.2.2.2 Flow Velocities and Minimum Slope 6-1
6.2.2.3 Pipe Size 6-3
6.2.2.4 Pipe Material 6-3
6.2.2.5 Pipe Cover 6-3
6.2.2.6 Curved Sewers 6-3
6.2.2.7 Change in Flow Direction 6-3
6.2.2.8 Extraneous Flows 6-3
6.2.2.9 Sewer Maintenance 6-3
6.2.3 Storm Manholes 6-3
6.2.4 Catch Basins and Gutters 6-4
6.2.4.1 Collection of Surface Runoff 6-4
6.2.4.2 Catch Basin Capacity 6-4

6.2.4.3 Catch Basin Construction 6-4
6.2.4.4 Gutters 6-4
6.2.5 Stormwater Pumping Stations 6-4

6.3 Stormwater Best Management Practices (BMPs) 6-4
6.3.1 Introduction 6-4
6.3.1.1 General 6-4
6.3.1.2 Design Criteria for Stormwater Quality Control 6-5
6.3.2 Source Control BMPs 6-5
6.3.3 Lot-Level BMPs 6-5
6.3.3.1 Reduced Lot Grading 6-5
6.3.3.2 Surface Ponding and Rooftop Storage 6-7
6.3.3.3 On-lot Infiltration Systems 6-8
6.3.3.4 Sump Pumping of Foundation Drains 6-9
6.3.4 Stormwater Conveyance System BMPs 6-10
6.3.4.1 Pervious Pipe Systems 6-10
6.3.4.2 Pervious Catch basins 6-11
6.3.4.3 Grassed Swales 6-12
6.3.5 End-of-Pipe Stormwater BMPs 6-14
6.3.5.1 Wet Ponds 6-14
6.3.5.2 Dry Ponds 6-17
6.3.5.3 Constructed Wetlands 6-19
6.3.5.4 Infiltration Trenches 6-22
6.3.5.5 Infiltration Basins 6-24
6.3.5.6 Filter Strips 6-25
6.3.5.7 Sand Filters 6-26
6.3.5.8 Oil/Grit Separators 6-27
6.3.6 BMP Screening and Selection 6-28
6.3.6.1 Initial Screening 6-28
6.3.6.2 Physical Constraints 6-28
6.3.6.3 Final Screening 6-28
6.3.6.4 Water Quality Control and Enhancement Opportunities 6-29

7.0 Wastewater Systems - Operating and Monitoring (Requirements and Guidelines) ... 7-1

7.1 Systems Operations 7-1
7.1.1 General 7-1
7.1.2 Reliability 7-1
7.1.3 Operations 7-1
7.1.4 Facility Classification And Operator Requirements 7-2
7.1.4.1 Facility Classification 7-2
7.1.4.2 Requirement For Having Certified Operators 7-2
7.1.4.3 Responsibility Of Operators 7-2
7.1.4.4 Responsibility of Facility Owners 7-2
7.1.4.5 Facilities Staffing Requirements: Certified Operators 7-5

7.2 System Monitoring 7-6
7.2.1 General 7-6
7.2.1.1 Sampling Procedures and Analytical Methods 7-6
7.2.1.2 Approval of Analytical Procedures 7-7
7.2.2 Operational Monitoring 7-7
7.2.2.1 Activated Sludge Systems 7-7

7.2.2.2 Wastewater Lagoons....7-9
7.2.3 Treatment Performance and Compliance Monitoring....7-9
7.2.3.1 Secondary Treatment - Mechanical (for current population < 20,000)....7-10
7.2.3.2 Tertiary Treatment - Mechanical (for current population > 20,000)....7-12
7.2.3.3 Aerated Lagoons (for current population < 20,000)....7-15
7.2.3.4 Aerated Lagoons (for current population > 20,000)....7-16
7.2.3.5 Wastewater Lagoons....7-18
7.2.4 Issue Oriented and Follow-up Monitoring....7-18
7.2.4.1 Activated Sludge System....7-19

7.3 Record Keeping....7-20

7.4 Reporting....7-20

APPENDIX A....A-1
APPENDIX B....B-1
APPENDIX C....C-1
APPENDIX D....D-1
APPENDIX E....E-1
APPENDIX F....F-1
APPENDIX G....G-1

FOREWORD

Alberta Environment (AENV) is responsible for the Drinking Water and Wastewater Programs for large public systems in Alberta. AENV considers the establishment of standards and guidelines for municipal waterworks, wastewater and storm drainage facilities an integral part of our regulatory program directed at ensuring public health and environmental protection. AENV's objective is to develop comprehensive and scientifically defensible standards and guidelines that are effective, reliable, achievable and economically affordable.

Regional Health Authorities (RHAs) are responsible for the application of the Public Health Act of Alberta within their Regional boundaries. The role of RHAs in the spirit of the Public Health Act, applies to all drinking water systems, both large and small, and to all aspects of safe drinking water production and delivery, if there is a concern about health impacts or disease transmission.

The system owners / utilities are responsible for meeting AENV's regulatory requirements and for the production and delivery of safe drinking water to the consumers. They are also responsible for maintaining water distribution system to the service connection, and will assist the home / building owners to identify any water quality issues within building plumbing. However, home / building owners are still responsible for plumbing repairs, system corrections and water quality within their building.

Engineering consultants and / or the system owners / utilities are responsible for the detailed project design and satisfactory construction and operation of the waterworks and wastewater systems.

In accordance with the Potable Water Regulation (277/2003) and the Wastewater and Storm Drainage Regulation and Wastewater and Storm Drainage (Ministerial) Regulation (119/93 and 120/93), a waterworks system, a wastewater system, and a storm drainage system shall be designed so that they meet, as a minimum, the performance standards and design requirements set out in the Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems, published by AENV, as amended or replaced from time to time, or, any other standards and design requirements specified by the Regional Director. AENV last revised its Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems in December 1997.

In the wake of the recent tragedies in Walkerton, Ontario and North Battleford, Saskatchewan, as a result of contaminated drinking water, jurisdictions across the country were compelled to re-evaluate their drinking water programs. Though Alberta has a sound drinking water program, and the events that led to tainted drinking water in Walkerton and North Battleford are unlikely to happen in Alberta, AENV commenced a stakeholder driven process in the fall of 2002 to review and revise the December 1997 standards and guidelines for waterworks systems. No update is done at this time on wastewater and storm drainage systems.

To facilitate an open and transparent process in the development of these standards and guidelines, AENV invited recognized waterworks experts within the province to participate in the initial development and drafting stage. Representatives from the municipalities, engineering consultants, academia and other government departments participated in an Advisory / Working Group to guide and direct AENV in this process.

In developing the drinking water program and the associated waterworks standards and guidelines, AENV adopted the universally accepted "multi-barrier source to tap approach".

AENV's position for the provision of safe drinking water is quite simple; there should be no weak link in the program that may potentially affect the quality of drinking water and increase the risk of someone becoming ill from drinking the water. Thus, all activities and/or system components that directly relate to public health protection are grouped under "Section 1 - Waterworks System Standards"; activities and/or system components that are not considered critical in providing safe drinking water are included under "Section 2 – Waterworks System Guidelines".

"Section 1 - Waterworks System Standards", details all the critical elements of the drinking water program and the associated design and/or performance standards. The key standards are based directly on USEPA's current surface water treatment rule. They are either narrative criteria or numerical limits for a number of specific parameters, design, construction and operation that ensure a particular environmental quality or public health objective. These are mandatory requirements with which owners are required to comply. If at any time, the performance standards are compromised, the approval / registration holder shall immediately report to the Regional Director, either:

1. by telephone at 1-780-422-4505; or
2. by a method:
    a. in compliance with the release reporting provisions in the Act and the regulations, or
    b. as authorized in writing by the Regional Director.

"Section 2 – "Waterworks System Guidelines" are intended to provide general guidance on how to achieve a certain level of system performance or reliability. Good engineering and best management practices are included under this section. These are not mandatory requirements, but they establish the minimum quality of the facility expected when the system owner / utility applies for an approval.

It is recognized that not all existing waterworks systems will meet the standards published in this document. As the new standards will have a direct impact on the safety of drinking water, Alberta Environment expects the upgrading to be done in a reasonable period. It is proposed that all waterworks systems that hold an approval or registration be upgraded to meet these new standards before April 1, 2012. The system owners / utilities are also expected to develop and submit to the Regional Director a five-year capital plan before April 1, 2007 to upgrade the system. The capital plan may be developed in conjunction with Alberta Infrastructure and Transportation if eligible for funding under the Municipal Water/Wastewater Partnership program.

As indicated, no changes are made at this time to the standards and guidelines for wastewater and storm drainage systems. Sections 3, 5, 7, 8 and 10 as it appeared in the 1997 edition have been re-numbered as Sections 3, 4, 5, 6 and 7 respectively. Sections 3 and 4 are standards and must be adopted by the system owners; sections 5 and 6 are guidelines and may be adopted at the discretion of the system owners; and section 7 outlines the operation and monitoring guidelines and requirements.

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DEFINITIONS / ABBREVIATIONS

AO	-	Aesthetic Objectives
AENV	-	Alberta Environment
AWWA	-	American Water Works Association
BDOC	-	Biodegradable Dissolved Organic Carbon
BNR	-	Biological Nutrient Removal
BPJ	-	Best Professional Judgement
BPR	-	Biological Phosphorus Removal
BPT	-	Best Practicable Technology
CBOD	-	Carbonaceous Biochemical Oxygen Demand at 5 days and 20 °C
CFID	-	Continuous feed and intermittent discharge
DAF	-	Dissolved Air Flotation
DBP	-	Disinfection By-product
DCS	-	Distributed Control System
DO	-	Dissolved Oxygen
DOC	-	Dissolved Organic Carbon
EPEA	-	Environmental Protection and Enhancement Act
F/M	-	Food to Microorganism ratio
G	-	Velocity Gradient
GCDWQ	-	Guidelines for Canadian Drinking Water Quality
GWUDI	-	Groundwater under the direct influence of surface water
HPC	-	Heterotrophic Plate Count
HRT	-	Hydraulic Retention Time
IFID	-	Intermittent feed and intermittent discharge
MAC	-	Maximum Acceptable Concentration
MLSS	-	Mixed Liquor Suspended Solids
NH_3-N	-	Ammonia nitrogen
NSF	-	National Sanitation Foundation
NTU	-	Nephelometric Turbidity Unit
ORP	-	Oxidation Reduction Potential
OU	-	Odour Unit
PLC	-	Programmable Logic Controllers
QA/QC	-	Quality Assurance/Quality Control
RBC	-	Rotating Biological Contactor
SAR	-	Sodium Adsorption Ratio
SBR	-	Sequencing Batch Reactor
SRT	-	Sludge Retention Time
TBOD	-	Total Biochemical Oxygen Demand at 5 days and 20 °C
TOC	-	Total Organic Carbon
TP	-	Total Phosphorus
TSS	-	Total Suspended Solids
TTHM	-	Total Trihalomethanes
UC	-	Uniformity Coefficient
UV	-	Ultraviolet

Average daily design flow (water and wastewater) - The product of the following:

- design population of the facility, and
- the greatest annual average per capita daily flow which is estimated to occur during the design life of the facility.

Co-op - An organization formed by the individual lot owners served by a waterworks system, wastewater system or storm drainage system.

Granular Filter Media:

1. Effective Size (D_{10}) - Size of opening that will just pass 10% of representative sample of the granular filter media.
2. Uniformity Coefficient - A ratio of the size opening that will just pass 60% of the sample divided by the opening that will just pass 10% of the sample.

Groundwater - All water under the surface of the ground.

Maximum daily design flow (water) - Maximum three consecutive day average of past-recorded flows, times the design population of the facility. If past records are not available, then 1.8 to 2.0 times the average daily design flow.

Maximum hourly design flow (water) - 2.0 to 5.0 times the maximum daily design flow depending on the design population.

Maximum monthly average daily design flow (wastewater) - The product of the following:

1. design population of the facility, and
2. the greatest monthly average per capita daily flow which is estimated to occur during the design life of the facility.

Owners - Owners of the waterworks or wastewater systems as defined in the regulations.

Peak demand design flow (water) - the maximum daily design flow plus the fire flow.

Peak wastewater design flow (wastewater) - The sum of the peak dry weather flow rates as generated by population and land use, and the rate of all extraneous flow allowances, as determined for the design contributing area (see Section 7.1.1).

Potable Water – As defined in the EPEA. Other domestic purposes in the EPEA definition include water used for personal hygiene, e.g. bathing, showering, washing, etc.

Sodium Adsorption Ratio - A ratio of available sodium, calcium and magnesium in the soil solution which can be used to indicate whether or not the accumulation of sodium in the soil exchange complex will lead to a degradation of soil structure.

$$SAR=\frac{Na}{[\frac{Ca}{2}+\frac{Mg}{2}]^{1/2}}$$

Note : All concentrations expressed in milliequivalents per litre

Surface water - Water in a watercourse.

Watercourse - As defined in the EPEA.

1.0 Waterworks Systems Standards

This section outlines all the elements of a drinking water program that shall be adhered to for the safe delivery of drinking water. This includes the standards for the following elements:

1. Water quality;
2. Water treatment and performance;
3. Facility design and operation;
4. Water treatment chemicals;
5. Water treatment plant waste;
6. Water transmission and distribution;
7. Water quality monitoring, record keeping and reporting;
8. Data quality assurance;
9. Facility Accreditation; and
10. Facility classification and operator certification.

1.1 Potable Water Quality Standards

Potable water in the waterworks system shall meet the health related concentration limits (Maximum Acceptable Concentrations) in the Guidelines for Canadian Drinking Water Quality, published by Health Canada, as amended or replaced from time to time, for the parameters listed in Section 1.10.3 of this document, with the exception noted in Section 1.5.2 (2).

The Regional Director at his discretion may establish more stringent limits for the parameters listed, or establish additional parameters not listed in Section 1.10.3.

If iron or manganese reduction is practiced, then the treated water concentration for iron and manganese shall be 0.3 mg/L and 0.08 mg/L respectively.

1.2 Potable Water Treatment Standards

The level of potable water treatment is dependent on whether the raw water is obtained from a surface supply, groundwater under the direct influence of surface water (GWUDI), or groundwater. Criteria detailed in Section 1.2.1.4, shall be used to determine whether or not a source is under the direct influence of surface water.

1.2.1 Surface Water and GWUDI

1.2.1.1 Filtration and Disinfection Requirement

All waterworks systems, with surface water or GWUDI source, shall be provided with filtration and disinfection. Filtration and disinfection together shall achieve a minimum of 3-log reduction of *Giardia* and *Cryptosporidium*, and 4-log reduction of viruses. However, based on the raw water cysts and oocysts levels, systems shall achieve higher reduction of *Giardia* and *Cryptosporidium* in accordance with the log reduction requirements shown in Table 1.1. New facilities that have not done the source water assessment at the time of application for approval shall achieve a minimum of 3-log reduction of *Giardia* and *Cryptosporidium*, and 4-log reduction of viruses until the source water assessment is completed. If a system so chooses not to conduct raw water assessment to determine the level of reduction, it shall meet or exceed the maximum 5.5 log reduction of *Giardia* and *Cryptosporidium*, and 4-log reduction of viruses.

In addition, chlorine residual (free, combined or total chlorine) of not less than 0.1 mg/L shall be maintained in the water distribution system.

Table 1.1: Log Reduction Required for Filtered Systems

Raw Water *Giardia* Levels (cysts / 100 L)[1]	Raw Water *Cryptosporidium* Level (oocysts / 100 L)[1]	Log Reduction
< 1	< 7.5	3.0 log
> 1 and < 10	> 7.5 and < 100	4.0 log
> 10 and < 100	> 100 and < 300	5.0 log
> 100	> 300	5.5 log

[1] For communities with population larger than 10,000, the levels are based on running annual average of monthly samples over a two year period.
For communities with population less than 10,000, that are triggered based on E.coli sampling (see Section 1.2.1.3), the levels are based on running annual average of quarterly samples over a two-year period.

1.2.1.2 Filtration Exemption

Where a groundwater source is determined to be under the direct influence of surface water (GWUDI) as per Section 1.2.1.4, and where the source water quality conditions are suitable to avoid filtration as determined by the Regional Director, inactivation of *Giardia, Cryptosporidium* and viruses may be achieved by disinfection only. Disinfection shall achieve a minimum of 3-log reduction of *Giardia* and *Cryptosporidium*, and 4-log reduction of viruses.

1.2.1.3 Source Water Monitoring Requirements for Filtered Systems

Systems serving a population of at least 10,000 shall sample for cysts and oocysts monthly for a period of two years. Increase the frequency of monitoring to once a week during the spring run-off periods.

Systems serving a population of less than 10,000 shall first monitor for E.coli at least every two weeks for a one-year period. For systems that use sources other than lakes or reservoirs, increase the frequency of monitoring to once a week during the spring run-off periods. No cysts and oocysts monitoring will be required under the following conditions:

1. Systems that use lakes or reservoirs as sources and that have an average E.coli concentration of less than 10/100 mL, based on all the samples in a one -year period; or
2. Systems that use sources other than lakes and reservoirs, and that have an average E.coli concentration of less than 50/100 mL, based on all the samples in a one- year period.

Systems meeting the above criteria shall be required to provide filtration and disinfection to achieve a minimum of 3-log reduction of *Giardia* and *Cryptosporidium*, and 4-log reduction of viruses.

Systems serving a population of less than 10,000, triggered into cysts and oocysts monitoring, will sample at least four times per year for a period of two years, or as determined by the Regional Director based on site specific conditions.

Test results below detection limits shall be considered zero in calculating the average value.

Cysts and oocysts shall be sampled, analysed and reported in accordance with the USEPA Method 1623, or as amended.

1.2.1.4 Criteria to Determine GWUDI

AENV characterizes groundwaters as one of two types:

- Groundwater; and
- Groundwater Under the Direct Influence of Surface Water (GWUDI).

A brief description of these two types of groundwater is as follows:

1. Groundwater

   A raw water supply which is groundwater means water located in aquifer(s) that are either isolated from the surface, or where the subsurface soils act as an effective filter that removes micro-organisms and other particles by straining and antagonistic effect, to a level where the water supply may already be potable but disinfection is required as an additional health risk barrier.

2. GWUDI

   A raw water supply, which is groundwater under the direct influence of surface water, means ground water having incomplete or undependable subsurface filtration of surface water and infiltrating precipitation.

   Refer to Appendix E entitled *Assessment Guideline for Groundwater Under the Direct Influence of Surface Water (GWUDI)* for determining whether a groundwater source is GWUDI. Note that a source determined to be GWUDI would require treatment equivalent to that required by a surface water source.

1.2.2 Groundwater

Groundwater systems shall provide disinfection to achieve a minimum of 4-log reduction of viruses, unless otherwise exempted by the Regional Director.

For groundwater systems exempted by the Regional Director to provide 4-log reduction of viruses, the system owner/operator shall undertake the following steps to demonstrate there is little risk of contamination of the source:

1. a one time only assessment of the hydrogeologic sensitivity of the aquifer; and
2. a sanitary survey of the groundwater system once every five years. A sanitary survey is an onsite review of the water source, facilities, equipment, operation and maintenance of the system for the purpose of evaluating the adequacy of such source, facilities, equipment, operation and maintenance for producing and distributing safe drinking water.

In addition, a disinfectant residual (total chlorine not less than 0.1 mg/L) shall be maintained in the water distribution system.

1.3 Potable Water Treatment Performance Standards

1.3.1 Filtration

1.3.1.1 Rapid Sand Filtration

In order to obtain *Giardia, Cryptosporidium* and virus reduction credits, outlined in Section 1.4.1, conventional and direct filtration systems, as shown in Figure 1.1, shall meet the following turbidity standards. In addition to the turbidity requirement, particle reductions standards may also be used, at the discretion of system owner. Particle counts, however, may be monitored to optimize filtration, either for filter-to-waste times, or for monitoring of spikes, or for low-level optimization down to 0.02 or 0.03 NTU.

1. Turbidity Reduction

   The treated water turbidity levels from individual filters shall be less than or equal to 0.3 NTU at all times. Exceedance of this limit is allowed up to 1 NTU for a cumulative period of 15 minutes per day per filter for discharge into the clear water tank.

   The treated water turbidity levels from individual filters shall be based on continuous measurements of the turbidity and recorded at no more than five-minute intervals (with an on-line turbidimeter) at a point upstream of the combined filter effluent line or the clear water tank.

2. Particle Reduction

   Particle counts (particles greater than 2 μm) from individual filters shall not exceed an absolute value of 50 particles/mL. Exceedance of this limit is allowed up to 200 particles/mL for a cumulative period of 15 minutes per day per filter for discharge into the clear water tank.

   The particle counts shall be based on continuous measurements of the particles and recorded at no more than five-minute intervals (with an on-line particle counter) at a point upstream of the combined filter effluent line or the clear water tank.

1.3.1.2 Slow Sand Filtration

In order to obtain *Giardia Lamblia, Cryptosporidium* and virus reduction credits, outlined in Section 1.4.1, slow sand filtration systems shall meet the following turbidity standards:

1. Turbidity Reduction

   The treated water turbidity levels from individual filters shall be less than or equal to 1 NTU at all times. Exceedance of this limit is allowed up to 3 NTU for a cumulative period of three hours per day per filter for discharge into the clear water tank.

   The treated water turbidity levels from individual filters shall be based on continuous measurements of the turbidity and recorded at no more than five-minute intervals (with an on-line turbidimeter) at a point upstream of the combined filter effluent line or the clear water tank.

1.3.1.3 Membrane filtration

Cysts / oocysts / virus reduction credit for membrane filtration system shall be based on product specific challenge testing and verified by direct integrity testing of the membrane, as described in the latest edition of the USEPA Membrane Filtration Guidance Manual. An independent third party in accordance with the criteria outlined in this guidance manual shall perform the challenge tests. The maximum removal credit shall be the lower of the two values established during the challenge test, or the maximum log removal value verified by the direct integrity test during the course of normal operation.

As a minimum standard, the membrane filtration systems shall meet the following turbidity standards, particle reduction standards and direct integrity test standards:

1. Turbidity Reduction

   The treated water turbidity levels from individual filter train shall be less than or equal to 0.1 NTU at all times. Exceedance of this limit is allowed up to 0.3 NTU for a cumulative period of 15 minutes per day per filter module for discharge into the clear water tank.

   The treated water turbidity levels from individual filter train shall be based on continuous measurements of the turbidity and recorded at no more than five-minute intervals (with an on-line turbidimeter) at a point upstream of the combined filter effluent line or the clear water tank.

2. Particle Reduction

   Particle counts (particles greater than 2 μm) from individual filter train shall not exceed an absolute value of 20 particles/mL. Exceedance of this limit is allowed up to 50 particles / mL for a cumulative period of 15 minutes per day per filter train for discharge into the clear water tank. For those membranes that operate under vacuum, where air entrainment may be a problem, exceedance of this limit is allowed up to 200 particles / mL for a cumulative period of 30 minutes per day per filter train.

   The particle counts shall be based on continuous measurements and recorded at no more than five-minute intervals (with an on-line particle counter) at a point upstream of the combined filter effluent line or the clear water tank.

3. Direct Integrity Test

   A direct physical test shall be applied daily to each membrane train to identify and isolate integrity breaches. The direct integrity test shall be applied in a manner such that a 2 um hole contributes to the response from the test. The direct integrity test shall be capable of verifying the log reduction value awarded to the membrane process.

1.3.1.4 Cartridge Filtration

Cysts / oocysts reduction credit for cartridge filtration system shall be based on product specific challenge testing, as described in the latest edition of the USEPA's LT2ESWTR Toolbox Guidance Manual. An independent third party in accordance with the criteria outlined in this guidance manual shall perform the challenge tests.

As a minimum standard, the cartridge filtration systems shall meet the following turbidity standards. In addition to the turbidity requirement, particle reductions standards may also be used, at the discretion of system owner. Particle counts, however, may be monitored to optimize filtration, either for filter-to-waste times, or for monitoring of spikes, or for low-level optimization down to 0.02 or 0.03 NTU.

1. Turbidity Reduction

   The treated water turbidity levels from individual filter module shall be less than or equal to 0.3 NTU at all times. Exceedance of this limit is allowed up to 1 NTU for a cumulative period of 15 minutes per day per filter module for discharge into the clear water tank.

   The treated water turbidity levels from individual filter module shall be based on continuous measurements of the turbidity and recorded at no more than five-minute intervals (with an on-line turbidimeter) at a point upstream of the combined filter effluent line or the clear water tank.

2. Particle Reduction

   Particle counts (particles greater than 2 μm) from individual filter module shall not exceed an absolute value of 50 particles/mL. Exceedance of this limit is allowed up to 200 particles / mL for a cumulative period of 15 minutes per day per filter for discharge into the clear water tank.

   The particle counts shall be based on continuous measurements and recorded at no more than five-minute intervals (with an on-line particle counter) at a point upstream of the combined filter effluent line or the clear water tank.

1.3.2 Disinfection

General Performance Requirements

All waterworks systems shall provide disinfection to:

1. inactivate the pathogens not removed by clarification and filtration, and achieve the level of cysts / oocysts reduction as stipulated in Table 1.1;
2. inactivate viruses in surface water, GWUDI, and groundwater systems, and achieve the level of virus reduction as stipulated in Sections 1.2.1 and 1.2.2; and
3. maintain total chlorine residual in the water distribution system.

1.3.2.1 Chlorine (Free and Combined)

CT Concept

Use of the "CT" disinfection concept shall be followed to demonstrate satisfactory treatment since monitoring for very low levels of pathogens in treated water are analytically very difficult.

The CT concept uses the combination of disinfectant residual concentration (mg/L) and the effective disinfection contact time (in minutes) at maximum hourly flows to measure effective pathogen reduction. The residual is measured at the end of the process, and the contact time used is the T_{10} of the process unit (time for 10% of the water to pass). Section 1.10.3.7 provides details on how T_{10} should be determined.

CT = Concentration (mg/L) x Time (minutes)

Additive CT Values (Disinfection Profiling)

Total log reduction by disinfection for a facility can be determined by summing the CT_{ratio} achieved in individual process units in series. CT_{ratio} is the ratio of CT_{actual} and $CT_{required}$. Different disinfectants can also be used in different process steps.

Determine the log reduction for the CT achieved, from the CT Tables in Appendix A and B.

Specific Performance Requirements

If free chlorine is used as the primary disinfectant, the CT required for the inactivation of cysts shall be in accordance with the CT Tables in Appendix A, and CT required for the inactivation of viruses shall be in accordance with the CT Tables in Appendix B.

If combined chlorine is used as the primary disinfectant, the CT required for the inactivation of cysts and viruses shall be in accordance with the CT Tables in Appendix B.

If free or combined chlorine is used as the primary disinfectant, the minimum chlorine concentration, at the location where C is measured for the calculation of CT, shall be 0.2 mg/L.

If combined chlorine is used as the primary disinfectant, chlorine should be added ahead of ammonia to have sufficient free chlorine to achieve the required removal of viruses.

Residual disinfectant concentration in the distribution system, measured as total chlorine, free chlorine, or combined chlorine shall be at least 0.1 mg/L in all of the samples taken.

Maximum residual disinfectant concentration, measured as free chlorine shall not exceed 4.0 mg/L, or as combined chlorine shall not exceed 3.0 mg/L, anywhere in the system.

1.3.2.2 Ultraviolet Light

UV Dose (based on the IT Concept that is equivalent to the CT Concept of a chemical disinfectant dose)

Use of UV Dose (the "IT" disinfection concept) shall be used to demonstrate satisfactory treatment since monitoring for very low levels of pathogens in treated water is analytically very difficult. The degree of inactivation of each target pathogen is based on the UV dose applied to the system. UV dose in ideal reactors is defined as the product of intensity of radiation I (milliwatt/cm^2), and the length of time (seconds) the water is exposed to UV radiation.

UV Dose = IT (mJ/cm^2) = Intensity (mW/cm^2) x Time (s)

For non-ideal field reactors, the delivered dose is validated by empirical methods that involve field reactor performance calibration against an ideal laboratory reactor.

Specific Performance Requirements

If UV light is used as a primary disinfectant, the dose required for the inactivation of cysts, oocysts and viruses shall be in accordance with the IT Tables in Appendix C.

If the UV dose is inadequate to achieve the required virus reduction, then UV shall be followed by another disinfectant such as chlorine with the appropriate CT to achieve the desired results with the target virus.

UV shall be always followed by a secondary disinfectant such as chlorine to maintain a residual in the water distribution system.

1.3.2.3 Chlorine dioxide

CT Concept

Same as for chlorine, see Section 1.3.2.1.

Specific Performance Requirements

If chlorine dioxide is used as the primary disinfectant, the CT required (Concentration C in mg/L x Contact Time T_{10} in minutes) for the inactivation of cysts and viruses shall be in accordance with the CT Tables in Appendix B.

Individual residual oxidants (chlorine dioxide, chlorite and chlorate) shall not exceed the MACs for these substances stipulated in the GCDWQ.

Chlorine dioxide shall be followed by a secondary disinfectant such as chlorine to maintain a residual in the water distribution system.

1.3.2.4 Ozone

CT Concept

As in the case of chlorine and chlorine dioxide, use of the "CT" disinfection concept shall be followed to demonstrate satisfactory treatment since monitoring for very low levels of pathogens in treated water are analytically very difficult.

Specific Performance Requirements

If ozone is used as the primary disinfectant, the CT required (Concentration C in mg/L x Contact Time T_{10} in minutes) for the inactivation of cysts and viruses shall be in accordance with the CT Tables in Appendix B.

Ozone shall be followed by a secondary disinfectant such as chlorine to maintain a residual in the water distribution system.

1.3.3 Fluoridation

When fluoridation is practised, adequate controls shall be maintained at all times to provide a fluoride ion concentration in treated water to meet the optimum concentration in the latest edition of GCDWQ (0.8 to 1.0 mg/L in the current web edition). A monthly average and daily variation shall be within ± 0.1 mg/L and ± 0.2 mg/L respectively.

The Regional Director, under certain circumstances, may allow optimum concentration to be lower than 0.8 mg/L.

1.4 Potable Water Treatment Credits

1.4.1 Filtration Credit

If treated water turbidity or particle count meets the prescribed limits in Section 1.3.1, the inactivation credit for *Giardia*, *Cryptosporidium* and viruses through filtration shall be determined in accordance with Table 1.2.

Table 1.2: Giardia, Cryptosporidium and viruses reduction credit through filtration

Technology	Cyst/oocyst credit	Virus credit
Conventional filtration	3.0 log	2.0 log
Direct filtration	2.5 log	1.0 log
Slow sand or diatomaceous earth filtration	3.0 log	2.0 log
Microfiltration, ultrafiltration and membrane cartridge filtration	Removal efficiency demonstrated through challenge testing and verified by direct integrity testing	No credit
Nanofiltration and reverse osmosis	Removal efficiency demonstrated through challenge testing and verified by direct integrity testing	Removal efficiency demonstrated through challenge testing and verified by direct integrity testing
Microfiltration, ultrafiltration, nanofiltration, reverse osmosis and membrane cartridge filtration, preceded by coagulation, flocculation and sedimentation	Minimum 3.0 log if removal efficiency demonstrated through challenge testing and verified by direct integrity testing	Minimum 2.0 log if removal efficiency demonstrated through challenge testing and verified by direct integrity testing
Cartridge filtration	Removal efficiency demonstrated through challenge testing. Maximum 2 log credit given with demonstration of at least 3 log removal efficiency in challenge testing	No credit

1.4.1.1 System Non-compliance with respect to turbidity reduction

For systems not meeting the turbidity or particle requirements outlined in Section 1.3.1, AENV may grant a limited or no filtration credit for a limited period till the plant can be optimized or upgraded. Plants with no filtration credit shall provide disinfection to achieve at least 3-log reduction of *Giardia* cysts, 3-log reduction of *Cryptosporidium* oocysts and 4-log reduction of viruses, or issue boil water advisories to the consumers served by the plants. [Note: Filtration systems that are unable to achieve a finished water turbidity as outlined in Section 1.3.1, will be required to implement a system evaluation program, and develop an optimization or upgrading plan. System evaluation program will include procedures to determine the log reduction requirement of Giardia cysts and *Cryptosporidium* oocysts].

1.4.2 Disinfection Credit

Depending on the disinfectant used, the inactivation credit for *Giardia, Cryptosporidium* and viruses through disinfection shall be determined in accordance with the CT Tables in Appendices A or B, or the IT Table in Appendix C.

1.4.3 Other Options Credit

Depending on the raw water quality, filtration alone may not be adequate to achieve the inactivation required for cysts and oocysts. Table 1.3 includes the options available to the system owner for further inactivation of cysts and oocysts. The intent of this approach is to provide systems with flexibility in selecting cost effective compliance strategy to achieve the required log reduction.

Table 1.3: Giardia, Cryptosporidium and viruses reduction credit through other options (Additional to the credit in Table 1.2)

Options	Treatment Credit with Design and Implementation Criteria
Pre-sedimentation basin with coagulation	0.5 log credit for cysts and oocysts with continuous operation and coagulant addition. Basin shall achieve 0.5 log turbidity reduction based on monthly mean of daily measurements in 11 of the 12 previous months. All flows shall pass through the basins
Two-stage lime softening	0.5 log credit for cysts and oocysts for two-stage softening with coagulant addition. Coagulant shall be present in both clarifiers and includes metal salts, polymers, lime, or magnesium precipitation. Both clarifiers shall treat 100% of flow
Combined filter performance; or Individual filter performance	0.5 log credit for cysts and oocysts if combined filter effluent turbidity is less than 0.15 NTU in 99% of measurements made in a calendar month; or 1.0 log credit for cysts and oocysts if individual filter effluent turbidity is less than 0.1 NTU in 99% of measurements made in a calendar month
Chlorine	Log credit for cysts and viruses on demonstration of compliance with CT tables in Appendices A and B
Chlorine dioxide	Log credit for cysts and viruses on demonstration of compliance with CT tables in Appendix B
Ozone	Log credit for cysts and viruses on demonstration of compliance with CT tables in Appendix B
UV light	Log credit for cysts, oocysts and viruses on demonstration of compliance with IT tables in Appendix C

1.5 Potable Water Treatment Facility Design Standards

1.5.1 Surface Water and GWUDI

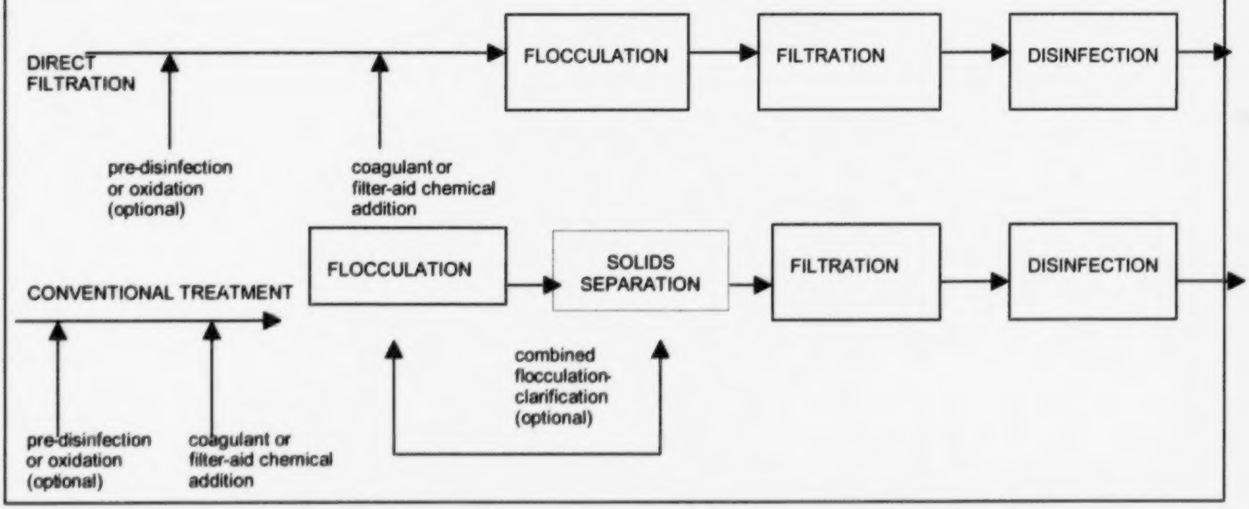
This section identifies the design requirements for those technologies acceptable for treatment of surface water and GWUDI. What are included in this section are minimum requirements to ensure reliable and safe drinking water supplies. These design requirements should be considered in conjunction with acceptable design practices and specific design criteria outlined in Section 2 – Waterworks Systems Guidelines.

1.5.1.1 Rapid Sand Filtration

The rapid sand filtration system shall be designed to produce water that meets the drinking water quality and the minimum performance requirements for rapid sand filtration outlined in section 1.3.1.1. Chemically assisted rapid sand filtration of water is a multi-step treatment process that includes chemical mixing, coagulation, flocculation, solids separation and filtration (conventional treatment); or chemical mixing, coagulation, flocculation and filtration (direct filtration), as illustrated in Figure 1.1.

Direct filtration may be used for source water that is consistently low in turbidity, colour and dissolved organic carbon. Water shall be suitable for charge neutralization with low coagulant dosages with the goal of forming a pinpoint sized floc that is filterable, rather than a settleable floc. Direct filtration will be acceptable only if it can be substantiated by undertaking a pilot study using the proposed source water.

FIGURE 1.1
TYPICAL WATER TREATMENT PROCESSES



DIRECT FILTRATION
FLOCCULATION
FILTRATION
DISINFECTION
pre-disinfection or oxidation (optional)
coagulant or filter-aid chemical addition
CONVENTIONAL TREATMENT
FLOCCULATION
SOLIDS SEPARATION
FILTRATION
DISINFECTION
combined flocculation-clarification (optional)
pre-disinfection or oxidation (optional)
coagulant or filter-aid chemical addition

The filtration systems shall be designed to produce water that meets the potable water quality standards (section 1.1), potable water treatment standards (Section 1.2), and the potable water treatment performance standards (1.3.1.1). The systems shall meet the following requirements:

1. Chemical Mixing and Coagulation

Chemical mixing and coagulation shall be achieved either in a separate process tank or with an in-line mixing device.

2. Flocculation

To prevent short-circuiting and to permit defined zones of reduced energy input, the flocculation process shall have two or more stages / compartments unless specifically exempted by AENV. A minimum of two flocculation trains shall also be provided unless specifically exempted by AENV.

3. Solids Separation

Solids separation shall be achieved either through sedimentation or through flotation. At least two sedimentation basins or two flotation basins shall be provided unless specifically exempted by AENV.

4. Filtration

Filtration represents the final particulate removal step in water treatment. Rapid sand filtration shall be allowed only in conjunction with chemical pre-treatment of water.

Any sudden increase or decrease on a dirty filter will cause some detriment to filtered water quality for a brief period, for this reason the filters shall be designed for continuous operation.

Filters shall be gravity feed type. Each filter shall be equipped with an on-line turbidimeter with a recorder for continuous monitoring of effluent quality.

a. Filtration Rate

Filter loading rate shall be considered a key factor in the design of a filtration system from both the filtrate quality and filter-run points of view. The filtration rate is dependent on the quality of raw water, the extent of pre-treatment and the characteristics and depth of the filter media. High-rate filtration (> 9 m/h) will be acceptable only if undertaking a filter column study using the proposed source water can substantiate it.

b. Filter to Waste

Water treatment plants shall be designed with filter-to-waste provision; piping shall be designed for the capacity of that filter. Precautions should be made to prevent backflow from the filter-to-waste stream to any component of the potable water supply system. The design should provide for switching filter-to-waste flow to filter-on-line (water production) without changing velocity through the filter to reduce turbidity spikes.

Duration of the filter-to-waste cycle must be tied to the actual turbidity of the wasted water or to a pre-determined time for the turbidity of the wasted water to reach the required limit.

c. Number of Filters

The number of filter units may vary with the plant capacity. For plants with capacity greater than 150 m^3/d, a minimum of two filters shall be provided, each capable of independent operation and backwash. Where possible, three filters or more shall be provided; except during repair/emergency, all filters shall be operational. Where only two filters are provided, each filter should have a hydraulic capacity not less than 150% of design filtration rate.

d. Filter Backwash

Filter backwash shall be up-flow water wash, or up-flow water wash with surface wash, or up flow water wash with air scour. Declining rate backwash systems are not acceptable.

1.5.1.2 Slow Sand Filtration

The slow sand filter system shall be designed to produce water that meets the drinking water quality standards and minimum performance requirements for slow sand filtration outlined in Section 1.3.1.2.

1. Filtration

Fluctuating load on the filter upsets the schmutzdecke, resulting in poor quality of filtered water for brief periods. For this reason the filters shall be designed for continuous operation. Each filter shall be equipped with an on-line turbidimeter with a recorder for continuous monitoring of effluent quality.

a. Filtration rates

Filter loading rate is a key factor in the design of slow sand filtration system from both the filtrate quality and filter run lengths points of view. The percent removal of turbidity generally declines with increasing loading rate. Unless specifically exempt by AENV, the optimum filtration rates shall be ascertained by pilot testing over the annual cycle.

b. Filter to Waste

During the filter ripening period, following the start-up of a new filter or a re-built filter bed, filtered water may be of very poor quality. The ripening period typically ranges from about one week to several months. The filters will not meet the minimum performance requirements during the ripening period and the water produced during this period shall be wasted or recycled to the pre-treatment works. Water treatment plants shall be designed with this filter-to-waste provision. Precautions should be made to prevent backflow from the filter-to-waste stream to any component of the potable water supply system.

During the filter-to-waste mode, the filter may be operated at high hydraulic loading rates and wasted water turbidity shall be measured daily until it reaches the acceptable level.

c. Number of Filters

Slow sand filters shall have a minimum of two or more cells so that when one is out of service for scraping or other reasons, another filter bed can continue producing sufficient amounts of water for the community.

Because of the prolonged filter ripening period, systems with two cells shall have each cell capable of producing maximum daily design flow for the community. Systems with more than two cells shall have all cells but one capable of producing maximum daily design flow.

1.5.1.3 Membrane Filtration

Membrane filtration shall be a pressure-driven or vacuum-driven process and remove particulate matter larger than 1 micron (um) using a non-fibrous engineered barrier. The process shall also have a measurable removal efficiency of a target organism that can be verified through the application of a direct integrity test.

From municipal drinking water point of view, only Micro Filtration System (MF) and Ultra Filtration System (UF) are of interest. This section outlines the design requirements for MF and UF systems.

A Membrane Cartridge Filtration device that utilizes a membrane filtration media capable of removing particulate matter larger than 1 micron, and which can be subjected to direct integrity testing would also satisfy the requirements for membrane filtration.

1. Treatment Objectives

It is critical that the treated water quality objectives are defined prior to the membrane process selection.

MF and UF membrane systems typically used for surface water treatment or GWUDI can only remove the non-dissolved portion of organic carbon. Dissolved organic removal is typically achieved through pretreatment. Pretreatment chemicals and the organic carbon itself can be contributors to membrane fouling and therefore pre-treatment processes should be evaluated carefully. Groundwater containing contaminants such as iron, manganese and arsenic may also be treated with membrane filtration.

2. Design Flow Rate

The design flow rate for membrane systems is the net filtered output desired from the membrane system. This shall take into account the loss of feed water used for backwashing and/or rejected from the system (waste stream) and the lost production while a unit or train is out of service for chemical cleaning.

Recovery Rate is the membrane system's final product volume over a given time period divided by the feed water flow volume. In order to achieve the lower recovery rate mode, the raw water feed system and waste-handling systems shall be sized to handle the larger flows.

The design flow rate per membrane train is a function of the total membrane surface area in the module and the flux rate selected. The membrane surface area per train is a physical characteristic dependant on the membrane manufacturer chosen. The flux rate shall be selected in consultation with the membrane manufacturer and the characteristics of the source water being treated. Optimum flux rates should be selected based on pilot results considering the required design production rates with both cold and warm water. Cold water can significantly reduce the flux rate of a membrane system; hence the seasonal demands should be carefully evaluated.

3. Membrane Selection

The style of membrane chosen should best address the feed water conditions, integration with other treatment, hydraulic limitations, residuals, available space and required capacity. Cartridge (pressure) or submerged (vacuum) type systems may be used. Select a membrane manufacturer that adopts and maintains the philosophy of "universal configurations" in order to accommodate future upgrades and products.

4. Piloting

The suitability of using a membrane system for a particular raw water source shall be substantiated or verified by conducting a pilot or demonstration study under "worst case" situations. The objective of the pilot or demonstration study is to assess the performance and reliability of the membrane system during the critical raw water conditions. The study shall also determine how the system would operate as an integrated process, react to various levels of pretreatment, and assess the fouling potentials. Sufficient data shall be collected to select the most economical design flux rate, cleaning regime and cleaning frequency for the full-scale system.

Prior to conducting a pilot study, a plan shall be developed and submitted to AENV for review. The plan shall include, but not be limited to the following information:

a. Raw water quality data for at least the previous 12-month period highlighting the variations in raw water quality, temperature and seasonal demands.

b. Proposed piloting schedule including rationale based on an evaluation of the historical raw water quality and seasonal variations. Critical water quality conditions may include, but not be limited to: high turbidity, cold water, high organics, and the presence of algae.

c. A schematic of the proposed pilot set up along with descriptions of the proposed processes and facilities. A comparison should be included of the proposed full-scale facility with any differences noted and discussed.

d. A description of the modes of operation to be tested and the parameters to be evaluated. For example: cold and warm temperature flux, percent recovery, trans-membrane pressure, pre-treatment requirements, backwash frequency, backwash duration, backwash method, clean-in-place method and frequency, and post treatment.

e. Time schedules for each mode of operation. The schedule should address the relationship between the modes of operation and how they relate to the critical water quality conditions.

f. Parameters to be monitored, including sampling locations, frequency, and method of monitoring or conducting analytical testing. Feedwater quality parameters are to include, but not be limited to: turbidity, TOC, DOC, UV254 absorbance, algae, calcium, magnesium, iron, manganese, total hardness, total alkalinity, pH and temperature.

g. Quality assurance/quality control procedures to be used (including information on the accredited laboratory to be used).

h. Description of the analysis to be used for evaluating the data collected.

i. A description of proposed challenge tests to substantiate the specific pathogen log removal credit requested by AENV.

j. The proposed plant design capacity ratings as a function of water temperature and other flux rate limitations.

k. The pilot study shall be run for a sufficient period to obtain meaningful design data. The selection of the test period(s) shall be based on the historical raw water quality data and critical conditions.

The proponent shall prepare the pilot study report documenting the data collected; the analyses performed and summarize the findings and performance recommendations. Recommendations regarding cleaning frequency, integrity testing, maintenance recommendations, anticipated membrane service life and procedures to optimize membrane performance and longevity shall be included. The pilot report shall be submitted to AENV for review.

AENV, at its discretion, may relax or waive the pilot testing requirements where at least one year of acceptable full scale operational data exists from a water treatment plant which uses membranes similar to what is proposed, treats water of similar quality and under similar operating conditions using the same source. The proponent shall provide an in depth report of this full-scale plant data to AENV for review.

5. Pretreatment

Membrane filtration systems can be successfully integrated into various stages of an existing water treatment facility to replace, enhance or provide additional treatment barriers. The determination of the optimal location for integrating a membrane system within an existing treatment facility depends on site and project specific factors.

New or existing pre-treatment systems upstream of the membranes can affect the design of the membrane process. The pre-treatment unit processes that should be evaluated for the integration of membrane filtration systems are as follows:

a. Screening

Provide pre-screening of any membrane system to protect the membranes from damage by debris. The required screen size is a function of the raw water source and quality and the membrane manufacturer's requirements.

b. Oxidation

Oxidation can be integrated with membrane processes to assist with organics (TOC & DOC) and taste and odour reduction. It's recommended that the oxidation process be introduced as far upstream of the membrane process as possible. When using potassium permanganate care should be taken to avoid overdosing. Un-reacted or excess potassium permanganate can affect membrane performance by fouling the membranes or precipitating manganese on the downstream side of the membrane fibres. In addition to AENV's overall approval requirements, the proponent shall obtain approval / sign-off for the use of an oxidant from the membrane manufacturer.

c. Adsorption

Adsorption processes can be integrated with membrane process for removal of organics (TOC and DOC) and taste and odour causing compounds. In addition to AENV's overall approval requirements, use of any adsorbent shall be approved by the membrane manufacturer if used upstream of the membrane filters. Where powdered activated carbon (PAC) is proposed, the specific type of PAC should be approved.

d. Coagulation

Coagulation or enhanced coagulation upstream of the membrane process is typically practiced to precipitate dissolved organic substances (DOC). Pre-treatment coagulation is ideally incorporated into a flocculation or clarification process ahead of the membrane process however with some membrane systems the coagulation process can be combined directly into the membrane reactor vessel. In addition to AENV's overall approval requirements, the membrane manufacturer shall approve the use of any coagulant.

e. Clarification/Sedimentation

Clarification or sedimentation upstream of a membrane process is not always necessarily required. However, utilizing one of the two processes to reduce the solids loading on the membranes can improve membrane performance which could in turn lead to less membrane surface area required and extended life of the membranes. AENV and the membrane manufacturer must approve any upstream processes.

6. Number of Process Trains

A minimum of two independent membrane filter trains shall be provided. When determining the number of additional trains, the equipment turndown limitations, instrumentation limits and the range of seasonal flow variations anticipated shall be considered. Ensure that complete trains can be brought on or taken off line as required. When determining the total amount of membrane area and number of membrane trains to meet system demands, the effect of having one train off-line shall be taken into account. When a train is off-line for cleaning the remaining trains will need to be capable of operating at a higher flux rate (filtration rate) for the duration of the cleaning cycle in order to meet system demands.

For staged plant capacity, the appropriate infrastructure for the projected future demands should be installed. The piping, valves, tankage and pumping units required for the future demands can be either installed initially or, as a minimum, provisions made for the future trains. The purchase of the additional membrane capacity, however, can be deferred until the future demands are approached and be incrementally installed. Installing excess membranes too far in advance is not recommended due to the limited membrane life expectancy of an installed unit, even if this unit is not in service. The contract with the manufacturer shall ensure the availability of compatible membranes at such time as they are required.

7. System Configuration

System configuration options will depend on the membrane selected. The general arrangements shall provide safe and convenient access to the membrane modules and ancillary equipment for routine inspection, maintenance and repairs with sufficient clearances and means for membrane removal and replacement.

8. Ancillary Equipment Requirements

a. Feed Water or Permeate Pumps & Blowers

Where pumps and air blowers are employed, the number of duty pumps and air blowers required will depend on the number of process trains selected and the anticipated range of flows. A standby unit shall be installed and piped accordingly as a common standby unit for any process train in the event one of the duty units is out of service for maintenance or repair.

b. Isolation Valves & Unions

Isolation valves are required for each individual membrane assembly. The size of the individual modules is such that it is often impractical to isolate individual membrane modules. Instead, isolation valves are to be provided to isolate individual trains [and] membrane assemblies, or subsections of the membrane assemblies

c. Piping and Automated Valves

Some membrane systems operate over a wide range of pressures and have a significant number of automated valves. Select piping materials, restraints, and actuator speed controls suitable for the intended materials, service and to prevent water hammer.

d. Chemical Feed Systems

Chemical feed systems shall have standby pumping units. Refer to Section 1.7 "Potable water Treatment Chemicals Standards" for storage and safe handling of the chemicals.

9. Flow meter

Flow meters shall be provided on:

a. The main raw water supply line (or individual train raw water supply lines) to measure the feedwater volume entering the membrane system and for flow pacing of any pretreatment chemicals;

b. Individual permeate lines from each membrane train to measure the filtration rate and volume of each train and pace post disinfection chemicals;

c. Individual reject or concentrate lines from each train to measure the flow rate and volume of waste stream water for calculating the overall recovery rate of the train;

d. Individual backwash lines (or use of the permeate flow meters) to measure the backwash flow rate and volume;

e. The combined filter effluent line and / or the distribution main header leaving the plant; and

f. Dosing of pre-treatment chemicals and determination of system/unit production rate and recovery.

10. On-line meter

On-line turbidimeter shall be provided on the common feed water line to the membrane trains.

Both on-line turbidimeter and particle counting instruments shall be provided on the permeate discharge from each membrane train. Sample point connections should be provided at each rack or cassette for connection of a portable particle counter to aid in trouble shoot testing in the event of a broken fiber.

Provisions shall be made for pH and residual measurement, either online or at convenient sample points, on each membrane CIP tank to monitor the cleaning solution concentrations.

Pressure gauges and transmitters are also required on each membrane train to measure transmembrane pressures for monitoring the rate of fouling and backpulse pressures to avoid over pressurization and damage to the membrane fibers.

11. Backwashing and Cleaning

Backwashing (back-pulsing) and chemical cleaning frequencies, durations and procedures shall be obtained from the membrane manufacturer, based on pilot study data or similar application data.

Provide neutralization of the cleaning solutions either directly in the process tank where the CIP has taken place, or the solutions should be transferred into a holding tank to ensure sufficient time for neutralization and monitoring prior to discharge.

12. Residuals (Waste) Stream Disposal

Alberta Environment shall be consulted, as early as possible, when considering the use of membrane technologies, to determine the most suitable options for disposal of waste streams from both pilot scale and full scale membrane plants.

13. Integrity Tests

There are five key aspects for achieving an integral membrane system:

a. performance requirements (meeting the continuous removal of particles greater than one micron in diameter),
b. type of integrity test,
c. integrity test criteria and settings (pressures, alarm settings, etc.),
d. frequency of integrity testing (daily, weekly, etc.), and
e. management of the process and information (data management, shutdown procedures, etc.).

There are two basic types of integrity testing: continuous indirect monitoring and periodic direct integrity testing. Indirect integrity monitoring includes online particle counting used as a continuous indication of the membrane integrity. In general, sustained particle counts are expected to remain below 20 cts/mL in the filtrate. If filtrate particle counts jump above 20 cts/mL for an extended period, this may be an indication that a membrane fibre has been breached and shall be isolated and checked for integrity.

Direct integrity testing includes such methods as pressure decay, vacuum hold, bubble point, and sonic testing. Direct integrity testing shall be performed at least once daily.

In order to receive the requested log removal credits, an integrity-testing program as outlined in the latest edition of the USEPA's Membrane Filtration Guidance Manual, for the proposed plant shall be submitted for approval.

14. Filter-to-Waste Provisions

For those systems that have to be tested on-line during production in the event of a membrane integrity breach, filter-to-waste option shall be provided for diversion to waste.

1.5.1.4 Cartridge Filtration

Cartridge filters are pressure driven separation processes that remove particles larger than one (1) micron using an engineered porous filtration medium, through either surface or depth filtration. Cartridge filters are allowed under the following conditions:

a. As a primary filter
System capacity: < 1.0 L/s
Source water: GWUDI
Disinfection: filtration followed by UV and Chlorine

b. As a secondary filter

Primary filter:	Rapid Sand Filtration or Slow Sand Filtration
Source water:	Surface
Disinfection:	filtration followed by chlorine

The system design and operation of shall be based on the latest edition of the USEPA's LT2ESWTR Toolbox Guidance Manual.

1.5.2 Groundwater

This section provides the design and operational requirements for groundwater systems.

1. Minimum Requirements

   As a minimum standard, well construction shall conform to the requirements of the Water Well Regulation.

   All new wells and potentially contaminated wells shall be disinfected in accordance with AWWA standard C654-97, which outlines the procedures for shock chlorination and bacteriological testing for the disinfection of wells supplying potable water.

2. Removal of Fluoride

   Naturally occurring fluoride up to a concentration of 2.4 mg/L is acceptable in distributed water. Distributed water with concentration greater than 2.4 mg/L shall be treated to reduce the level to 1.5 mg/L or lower, as determined by the Regional Director.

1.5.3 Disinfection

1.5.3.1 Chlorine / Chloramines

In the production of chloramines, the ammonia residuals in the finished water, when fed in excess of stoichiometric amount needed, shall be limited to inhibit growth of nitrifying bacteria.

1 On-Site Sodium Hypochlorite Generation Systems

As an alternative to chlorine gas or delivered commercial strength sodium hypochlorite, it is permissible to use a weak (0.7 to 0.9%) sodium hypochlorite solution generated on-site. The sodium hypochlorite can either be used for primary or secondary disinfection. The sodium hypochlorite solution is delivered to the treatment process using a system of positive displacement diaphragm metering pumps, peristaltic or centrifugal pumps, eductors or a loop system with metered flow control valves.

Redundancy shall be incorporated into the system to ensure that disinfection of the drinking water is not interrupted when any component of the largest unit is out of service.

a. Generation System Equipment Requirements

All generation equipment shall be NSF 61 approved

Sodium hypochlorite generation shall not be permitted outside the sodium hypochlorite room, which is defined as the room where sodium hypochlorite is generated and stored. The water softener and brine tank may or may not be located inside the sodium hypochlorite room.

i. Water Softener

The number and size of the water softener(s) will depend on the projected range of flows and the hardness of the make-up water. Water hardness used for generating and diluting the brine solution shall be less than 25 mg/L as $CaCO_3$, unless otherwise endorsed by the manufacturer.

ii. Water heater/chiller

A water heater or chiller may be required to adjust the temperature of the water used to make up and dilute the brine solution. The number and size of the water heater(s) or chiller(s) will depend on the anticipated range of flows, and their associated water temperatures. Incoming water shall be maintained between 15°C and 27°C, or as recommended by the generation system manufacturer.

iii. Brine Tanks

The brine tank shall be sized to accommodate the maximum day chlorine demand at the maximum flow rate. A standby brine tank shall be provided if the system does not have sufficient sodium hypochlorite storage for a minimum of three days of operation at maximum demand.

Brine solution is corrosive, therefore the material used for all associated storage tanks and piping shall be compatible with the solution. The brine solution shall be stored such that the temperature of the solution does not go below 5°C.

iv. Dosing pumps

Where pumps are employed, the number and size of duty pumps required will depend on the chlorine demand and the anticipated range of flows. At least one standby unit shall be installed and piped accordingly as a common standby unit in the event that one of the duty units is out of service.

v. Isolation Valves and Unions

Sufficient isolation valves and unions shall be provided to allow the removal of any component of the generation equipment for maintenance or repair work without having to take the entire system off line.

b. Hydrogen Gas Byproduct Safety

A minimum of two air blowers shall be provided. When determining the number of additional blowers required, the size and number of generating units and storage tanks shall be considered. At least one standby unit shall be installed and piped accordingly as a common standby unit in the event that one of the duty units is out of service.

Blowers shall be required to dilute the hydrogen gas in the generator outlet piping and the headspace of the storage tanks. The blower shall be designed to achieve a hydrogen-in-air concentration of 1% or less, which corresponds to 25% of hydrogen's lower explosive limit (LEL). The blowers shall vent to outside of the building, and shall be interlocked with the generation process to ensure that sodium hypochlorite is only generated when there is adequate dilution airflow.

A minimum of two hydrogen gas detectors shall be placed in each room, for the generation equipment and the storage tanks. The hydrogen gas detectors shall be located at the highest points in each room or immediately over the generating equipment and storage tanks and be set to alarm at 50% of hydrogen's LEL (2% hydrogen in air) and be interlocked to shut down the generation system.

A flow switch and pressure switch shall be located in the dilution air ducting prior to the exterior vent. The flow switch and the pressure indicator shall be intrinsically safe.

c. Salt Supply and Storage

The salt (sodium chloride) supply shall be NSF 60 approved and also meet any further levels of purity required by the generation equipment manufacturer to ensure no negative impacts on the equipment or the quality of the finished water.

Salt shall be stored in a cool dry place where the humidity does not regularly exceed 75%.

Salt storage capacity shall be for a minimum 30 days at maximum day chlorine demand, unless exempted by the Regional Director.

d. Minimum Generation Capacity

The sodium hypochlorite dosing system shall have the capacity to be turned down to meet the minimum day demand, and not exceed the required "free" or "combined" chlorine residual.

The entire system shall have the capacity to be able to meet the maximum day chlorine demand at maximum plant flow rate, and maintain the required concentration of "free" or "combined" chlorine residual.

The system shall produce a sodium hypochlorite solution that is always below 1% as chlorine.

e. Sodium Hypochlorite Solution Storage Requirements

The sodium hypochlorite solution shall be stored in a location where the ambient temperature is maintained between 5°C and 37°C. A minimum of one day's chlorine storage is required if there is a 100% redundant generation system, equal to the capacity of the largest generation unit, or else a minimum of two day's storage is required along with the ability to use commercial sodium hypochlorite.

The sodium hypochlorite solution shall be stored such that it is not exposed to UV radiation.

The material of construction of the storage tank(s) shall be suitable for storing commercial grade NaOCL (10%-16%). All storage tanks shall be fully enclosed and all piping connections and personnel accesses shall be completely sealed from the building interior.

Level measurement devices (switches, transmitters etc.) shall be located in all storage tanks and shall be intrinsically safe.

f. Sodium Hypochlorite Quality Controls

The quality of the sodium hypochlorite shall be such that the required disinfection objectives are met and that it does not negatively impact the quality of the treated water.

g. Minimum Generation System Controls and Instrumentation

Control of the feed rate shall be:

1. automatic/ proportional controlled, whereby the fluoride feed rate is automatically adjusted in accordance with the flow changes to provide a constant pre-established dosage for all rates of flow; or
2. automatic/ residual controlled, whereby a continuous automatic fluoride analyzer determines the residual fluoride level and adjusts the rate of feed accordingly; or
3. compound loop controlled, whereby the feed rate is controlled by a flow proportional signal and residual analyzer signal to maintain a constant residual.

All systems shall be designed to have, water and brine flow indicator, cell level sensors, cell temperature sensors, tank level indicator, hydrogen dilution airflow pressure indication and dosing point residual indicator.

h. Electrolyzer Cleaning

Chemical cleaning frequencies, durations and procedures shall be in accordance with the recommendation made by the on-site generation system manufacturer.

Acidic cleaning solutions shall be neutralized prior to discharge.

i. Containment

Secondary containment shall be provided around the generation equipment, storage tanks and metering pumps. The containment shall include structures and valving to enable chemical neutralization prior to discharge.

j. Standby Provisions

Either a redundant generation system, equal to the capacity of the largest generation unit, or redundant storage to provide a minimum of two days sodium hypochlorite inventory at maximum chlorine demand and maximum plant flow rate, shall be provided.

The redundant storage shall allow for bulk commercial sodium hypochlorite storage. A mixing system including flow meters must be installed to enable the commercial sodium hypochlorite to be diluted to a 1% solution for proper dose control. The diluted product will be stored in the same storage tanks as the generated sodium hypochlorite is stored. All the necessary safety and ventilation systems shall be provided if the redundant storage approach is selected.

k. Residuals

The Regional Director shall determine the method of disposal of water softener regeneration waste, residual acid waste from cell cleaning, and any waste generated from tank cleaning.

l. Sodium Hypochlorite Generation Room Design Requirements

All rooms in which sodium hypochlorite is being generated shall be designed in accordance with the Alberta Building Code (occupancy, use etc.). If use of commercial sodium hypochlorite is provided in lieu of a redundant generating system, the design shall also comply with commercial sodium hypochlorite storage requirements.

1. Equipment Layout

Equipment layout will depend on the generation equipment selected, and the size of the system. The general arrangements shall provide safe and convenient access to the generator cells, brine tank, softener, storage tank, metering pumps and ancillary equipment for routine inspection, cleaning, maintenance and repairs with sufficient clearances and means for equipment removal and replacement. The brine tanks shall be located such that they facilitate salt delivery and minimize the impacts of salt dust during loading.

2. Ventilation

The sodium hypochlorite room ventilation system shall be completely separate from the building ventilation system and be capable of exhausting the air to atmosphere, outside of the building. The air-handling unit shall provide all the make-up air required for the hydrogen gas blowers, and the required ventilation. The quantity of outside air will be directly proportional to the number of dilution blowers operating.

Hydrogen dilution air outlets shall be above the roofline. The outlet shall be at least 150 mm above the roof and the point of discharge located to preclude contamination of air inlets to buildings or areas used by people. The vents to the outside shall have insect screens. Air inlets shall be louvered near the ceiling; the air temperature adjusted if required, as not to adversely affecting the sodium hypochlorite generation equipment.

3. Power Supply, Transformer/rectifier, for generation module.

The distance between the rectifier and the generation cells must meet the generation system manufacture's requirements.

Conductors and connectors shall be completely enclosed in protective insulation. No opening larger than 6 mm is permitted.

4. Room Access/Egress

Room access and egress shall be in accordance with the Alberta Building Code.

2. Chlorination Equipment Requirements

For all water treatment facilities, chlorine gas under pressure shall not be permitted outside the chlorine room. Chlorine room is the room where chlorine gas cylinders and/or ton containers are stored. Vacuum regulators shall also be located inside the chlorine room. Chlorine gas flow control and monitoring device may or may not be located inside the chlorine room.

For new and upgraded facilities, from the chlorine room, chlorine gas vacuum lines should be run as close to the point of solution application as possible. Injectors should be located to minimize the length of pressurized chlorine solution lines. A gas pressure relief system shall be included in the gas vacuum line between the vacuum regulator(s) and the chlorinator(s) to ensure that pressurized chlorine gas does not enter the gas vacuum lines leaving the chlorine room. The gas pressure relief system shall vent pressurized gas to the atmosphere at a location that is not hazardous to plant personnel; vent line should be run in such a manner that moisture-collecting traps are avoided. The vacuum regulating valve(s) shall have positive shutdown in the event of a break in the downstream vacuum lines.

As an alternative to chlorine gas, it is permissible to use sodium or calcium hypochlorite solution with positive displacement pumping. Anti-siphon valves shall be incorporated in the pump heads or in the discharge piping.

a. Capacity

The chlorinator shall have the capacity to dose enough chlorine to overcome the demand and maintain the required concentration of the "free" or "combined" chlorine.

b. Methods of Control

Chlorine feed system shall be automatic proportional controlled, or automatic residual controlled, or compound loop controlled.

Manual chlorine feed system may be installed for groundwater systems with constant flow rate.

c. Standby Provision

Standby chlorination equipment having the capacity to replace the largest unit shall be provided. For uninterrupted chlorination, gas chlorinators shall be equipped with an automatic changeover system. In addition, spare parts shall be available for all critical equipment.

d. Weigh Scales

Scales for weighing cylinders shall be provided at all plants using chlorine gas. At large plants, scales of the recording and indicating type are recommended. As a minimum, a platform scale shall be provided. Scales shall be of corrosion-resistant material.

e. Securing Cylinders

All chlorine cylinders shall be securely positioned to safeguard against movement. Tonne containers may not be stacked.

f. Chlorine Leak Detection

Automatic chlorine leak detection and related alarm equipment shall be installed at all water treatment plants using chlorine gas. Leak detection shall be provided for the chlorine rooms. Chlorine leak detection equipment should be connected to a remote audible and visual alarm system and checked on a regular basis to verify proper operation. Leak detection equipment shall not automatically activate the chlorine room ventilation system in such a manner as to discharge chlorine gas.

Chlorine leak detection equipment may not be required for very small chlorine rooms with an exterior door (i.e., floor area less than $3m^2$).

g. Safety Equipment

The facility shall be provided with personnel safety equipment to include the following: Respiratory equipment; safety shower, eyewash; gloves; cye protection; protective clothing; cylinder and/or toner repair kits.

Respiratory equipment shall be provided which has been approved under the Occupational Health and Safety Act, General Safety Regulation - Selection of Respiratory Protective Equipment. Equipment shall be in close proximity to the access door(s) of the chlorine room.

3. Chlorine Room Design Requirements

Where gas chlorination is practiced, the gas cylinders and/or the ton containers up to the vacuum regulators shall be housed in a gas-tight, well illuminated, corrosion resistant and mechanically ventilated enclosure. The chlorinator may or may not be located inside the chlorine room. The chlorine room shall be located at the ground floor level.

a. Ventilation

Gas chlorine rooms shall have entirely separate exhaust ventilation systems capable of delivering one (1) complete air change per minute during periods of chlorine room occupancy only - there shall be no continuous ventilation. The air outlet from the room shall be 150 mm above the floor and the point of discharge located to preclude contamination of air inlets to buildings or areas used by people. The vents to the outside shall have insect screens. Air inlets should be louvered near the ceiling, the air being of such temperature as to not adversely affect the chlorination equipment. Separate switches for fans and lights shall be outside the room at all entrance or viewing points, and a clear wire-reinforced glass window shall be installed in such a manner as to allow the operator to inspect from the outside of the room.

b. Heating

Chlorine rooms shall have separate heating systems, if forced air system is used to heat the building. Hot water heating system for the building will negate the need for a separate heating system for the chlorine room. Cylinders or containers shall be protected to ensure that the chlorine maintains its gaseous state when entering the chlorinator.

c. Access

All access to the chlorine room shall only be from the exterior of the building. Visual inspection of the chlorination equipment from inside may be provided by the installation of glass window(s) in the walls of the chlorine room. Windows should be at least 0.20 m^2 in area, and be made of clear wire reinforced glass.

There should also be a 'panic bar' on the inside of the chlorine room door for emergency exit.

d. Storage of Chlorine Cylinders

If necessary, a separate storage room may be provided to simply store the chlorine gas cylinders, with no connection to the line. The chlorine cylinder storage room shall have access either to the chlorine room or from the plant exterior, and arranged to prevent the uncontrolled release of spilled gas. Chlorine gas storage room shall have provision for ventilation at thirty air changes per hour. Viewing glass windows and panic button on the inside of door should also be provided.

In very large facilities, entry into the chlorine rooms may be through a vestibule from outside.

e. Scrubbers

For facilities located within residential or densely populated areas, scrubbers shall be provided for the chlorine room. The scrubbers shall be sized to handle leaks at least equal to full release of chlorine from the largest single container in the room.

1.5.3.2 Ultraviolet Light

1. General

For UV reactors of nominal diameter 300 mm and smaller, the system design shall follow the standards set out in the most recent of the:

- USEPA Ultraviolet Disinfection Guidance Manual; or
- German Association on Gas and Water Technical Standard W 294: UV Systems for Disinfection in Drinking Water Supplies – Requirements and Testing; or
- NWRI / AWWARF Ultraviolet Disinfection Guidelines for Drinking Water and Water Reuse.

For UV reactors greater than nominal diameter 300 mm, the system design shall follow the standards set out in the most recent edition of the *USEPA Ultraviolet Disinfection Guidance Manual.*

For privately owned development consisting of a single building, UV reactors certified under ANSI / NSF 55A may be used to provide the required disinfection.

Ultraviolet (UV) light disinfection (using 200-300 nm wavelength) is an acceptable form of treatment for *Cryptosporidium, Giardia* and bacteria, but it shall be used in conjunction with at least one other disinfectant (free chlorine, chlorine dioxide, or ozone) to provide the specified log reduction of viruses after filtration credits have been considered and a stable distribution system residual (free chlorine, chloramines).

A UV dose is applied by passing water through an inline reactor containing low or medium-pressure UV lamps contained within quartz sleeves, and with the lamp, baffle spacing and their configuration designed to meet the required application. The UV dose for a field reactor is a validated calculated dose derived from light intensity (I) readings from online UV sensors that are a function of power level to the lamps and lamp age; contact time (T) that is a function of flow rate and flow path through the reactor; the variable germicidal efficiency of different UV wavelengths; the absorption of different wavelengths by the water; and the impact of all of the above on the dose distribution produced by the reactor under the conditions of operation. The resulting dose distribution is the basis of determining the calculated effective UV dose delivered for any particular target organism, and from that dose, the effective logs reduction achieved. This calculated dose must however be validated to be in agreement with the empirically determined UV dose as determined in a bioassay.

2. UV System Location

The UV units shall be installed generally following filtration; this will provide some protection against fouling of the UV sleeve, and will utilize the water with the highest transmittance to UV in order to minimize the amount of UV equipment needed to deliver a target dose and also to minimize fluctuations in water quality that would require additional equipment to address the poorest of the water qualities. UV can be installed on a combined filter effluent or following individual filters. Reservoir effluent applications are less desirable because of the inability to monitor and address any "off-specification" water that might occur in the reservoir, and the greater risk of lamp/sleeve breakage at the higher water pressures (or high or low pressures from water hammer). Some UV lamps can run hot, and these units shall not be operated without water in the reactor (for medium pressure lamps, a temperature sensor is required to warn of any increase in temperature within the unit).

3. Validation

All UV models shall be validated full-scale (usually by the manufacturer through a third party). Validation is undertaken using acceptable test organisms, and with hydraulics that are defined as standard for such testing. Reactor validation establishes the acceptable operating range of the UV reactor and provides the information for design for specific sites. Daily monitoring confirms that performance continues to be met.

UV System Manufacturers shall validate performance using bioassay tests with organisms such as MS-2 bacteriophage or Bacillus subtilis spores for each UV system model. The dose-responses of each test organism should be determined in a collimated beam reactor (an ideal reactor). The bioassay results will be used to define the actual UV dose (Reduction Equivalent Dose or RED for the test organism) that is produced by the reactor being tested under the range of water quality and operating conditions over which the UV system was tested. Systems will need to demonstrate, the actual UV dose (Reduction Equivalent Dose or RED dose for the test organism) that is required to obtain the required RED for a particular target pathogen (e.g. it may require 30-40 mJ/cm^2 RED dose of a resistant test organism for a unit to meet the required 12 mJ/cm^2 RED dose for a less UV-resistant pathogen). The use of test organisms that have a similar or a slightly lesser UV sensitivity compared to the target pathogen will minimize the need for RED correction.

Validation of full-scale reactors shall be done on units with essentially the same hydraulic design, baffles and lamp layout as the full-scale installation and similar upstream and downstream piping and valving arrangements.

To obtain credit for UV disinfection, UV reactors shall be purchased from a supplier with formal third party reactor validation data that covers all the water quality and operating conditions to be encountered, and from which doses and organism inactivation levels can be determined for the relevant target design pathogens (e.g. *Giardia* & *Cryptosporidium*, enteric viruses, etc.). The validation certificate and/or report should list the acceptable range of operating conditions for the reactor (Min/Max Flow, UV %T range, lamp power levels, lamp age factor) under which the target design dose and target microbe inactivation levels can be achieved. It should also provide a full explanation of the manufacturers dose calculation to obtain the desired inactivation levels. Reactors can use low pressure, low pressure / high out-put, or medium-pressure UV lamps.

4. UV % Transmittance (T)

Knowledge of the UV_{254} absorbance/transmittance of the water to be treated is critical when designing for good performance of UV systems. Low UVT is usually due to high levels of dissolved organic material (DOC). Waters with UVT above 90% will usually work well with standard UV systems, 80-90% UVT will require more lamps and/or closer spacing, but waters with UVT less than 80% may require more design consideration and require appropriate validation of performance (it is more difficult to design with UVT less than 80%, since the power required to provide the required UV dose rises sharply as the %T decreases).

UV Percent Transmittance (UVT) is related to UV Absorbance (A) as per the following equation:
$UVT = 100 \times 10^{-A}$

Where the wavelength used is 254 nm, e.g. a UV_{254} absorption coefficient of 0.050 cm^{-1} = 89.1% (1 cm path length).

Design of UV systems should ideally be based on the worst-case water transmittance of at least 12 months of UVT data for each facility (e.g., using the 5^{th} percentile of monthly, bimonthly or weekly samples). If the UVT needs to be improved, then pre-treatment ahead of UV should be considered (enhanced coagulation, PAC or GAC treatment). UVT measurements should be performed on water, as it would enter the reactor (no lab filtration or pH adjustment.

5. Fouling

UV installations may also experience problems due to fouling of the lamp sleeves if levels of iron, manganese, hardness, alum, alkalinity or other chemical parameters are above recommended limits. LSI (Langelier Saturation Index) and CCPP (Calcium Carbonate Precipitation Potential) indices should be close to zero. UV units should have on-line mechanical sleeve cleaning devices or provision for physical-chemical cleaning; this will handle most normal levels of possible contaminants.

6. Power Supply

UV reactors (medium pressure lamps) are sensitive to fluctuating power supply, and can take 5-10 minutes to restart if power drops off (voltage fluctuations, brown-outs, momentary power-outs). Utilities should evaluate the reliability and quality of their power supply (if necessary backup power generators or UPS power supplies can be considered to limit the down – time of the power failure).

7. Chlorine Residual Reduction

If chlorine or combined chlorine residuals are present entering the UV reactor, the residual will decrease noticeably in passing the reactor. This effect is magnified if the water is receiving a higher UV dose e.g., due to turndown limits. Any chlorine, chloramines or permanganate residual present will decrease the UVT and such decreases should be incorporated into the design.

8. Broken Lamps/Mercury Spillage

Utilities shall develop an emergency response procedure for dealing with broken/cracked lamps/sleeves. The lamps contain a small amount of mercury, typically less than 1 gm per lamp and typically with modern lamp technology around 0.2 mg per watt of electrical power rating of the lamp. Any problems with lamp should be investigated to confirm whether breakage has occurred. The response for dealing with broken / cracked / lamps / sleeves should include notification of AENV, disposal of reactor water to waste where possible, and ongoing monitoring of water quality for mercury levels. One (1) gm of mercury dispersed in 1 ML of water would result in a 1 µg/L concentration (which is the current Health Canada Guideline MAC), so the risk from one lamp breakage is low. Multiple lamp breakages pose a higher risk. Lamp breakage offline (handling, storage, etc) should be handled with small mercury spill kits.

9. Maintenance Program

Utilities should develop a regular maintenance program to calibrate sensors, check lamps, quartz sleeves, lamp cleaning mechanisms, and to carry out UV lamp replacement as per the manufacturer's recommendation.

10. Contingency Plan

Utilities shall have in place a contingency plan for the possibility of total UV system outage.

11. Redundancy

The system shall be designed such that with the largest unit out of service, the remaining units shall provide the required disinfection.

1.5.3.3 Chlorine dioxide

Chlorine dioxide provides good cysts and virus protection but its use is limited by the restriction on the maximum residual of 0.5 mg/L ClO_2/chlorite/chlorate allowed in finished water. This limits usable residuals of chlorine dioxide at the end of a process unit to less than 0.5 mg/L. The effectiveness of chlorine dioxide also decreases with decrease in temperature.

Where chlorine dioxide is approved for use as an oxidant, the preferred method of generation is to entrain chlorine gas into a packed reaction chamber with a 25% aqueous solution of sodium chlorite ($NaClO_2$).

Warning: Dry sodium chlorite is explosive and can cause fires in feed equipment if leaking solutions or spills are allowed to dry out.

1.5.3.4 Ozone

Ozone is a very effective disinfectant for cysts, oocysts and viruses. Ozone becomes less effective in cold waters, which limits its application for oocyst inactivation under cold-water conditions. Ozone CT values shall be determined for the ozone basin alone; an accurate T_{10} value shall be obtained for the contact chamber, residual levels measured through the chamber and average ozone residual calculated. Ozone does not provide a system residual and should be used as a primary disinfectant only in conjunction with free and/or combined chlorine.

Ozone does not produce chlorinated by-products (such as trihalomethanes) but it may cause an increase in such by-product formation if it is fed ahead of free chlorine; ozone may also produce its own oxygenated by-products such as aldehydes, ketones or carboxylic acids. Any installed ozonation system shall include adequate ozone leak detection alarm systems, and an ozone off-gas destruction system.

Ozone may also be used as an oxidant for removal of taste and odour or may be applied as a pre-disinfectant.

Ozone production, operation and maintenance of ozonator shall be in accordance with USEPA's Alternative Disinfectants and Oxidants Guidance Manual – April 1999.

1.5.4 Fluoridation

1.5.4.1 General Requirements

In addition to the specific criteria outlined in this Section, all requirements specified under Section 1.7 – Potable Water Treatment Chemicals Standards should also be complied if fluoridation is practiced.

Any person proposing to add fluoride to a potable water supply shall apply for and obtain an approval or registration or an amendment to an approval or registration from AENV. This application shall contain the following information:

1. a copy of the bylaw of the municipal council, which provides the authority to fluoridate;
2. the number of the appropriate approval or registration issued with respect to the existing municipal plant; and
3. an engineering report, including:
    a. a description of the proposed fluoridation equipment,
    b. a statement identifying the fluoride compound that is proposed to be added,
    c. a description of chemical storage and ventilation,
    d. a description of the water metering used at the water treatment plant,
    e. the generic name of the chemical to be used as the source of fluoride ion, and its fluoride content,
    f. a current chemical analysis of the fluoride content of the raw water,
    g. the name and qualifications of the person directly responsible for the operation of the proposed fluoridation process,
    h. the type of equipment proposed at the water treatment plant to determine the fluoride concentration of the water, and
    i. the description of the testing procedure to be used to determine the fluoride level of the water.

1.5.4.2 Chemical Feed

The equipment used for feeding the fluoride to water shall be accurately calibrated before being placed in operation, and at all times shall be capable of maintaining a dose within 5% of the rate at which the machine is set.

The following chemical feed practices apply:

1. Where a dry feeder of the volumetric or gravimetric type is used, a suitable weighing mechanism shall be provided to check the daily amount of chemical feed;
2. Hoppers should be designed to hold a 24-hour supply of the fluoride compound and designed such that the dust hazard to operators is minimized;
3. Vacuum dust filters shall be installed with the hoppers to prevent dust from rising into the room when the hopper is filled;
4. Dissolving chambers are required for use with dry feeders, and the dissolving chambers shall be designed such that at the required rate of feed of the chemical the solution strength will not be greater than 1/4 of that of a saturated solution at the temperature of the dissolving water. The construction material of the dissolving chamber and associated piping shall be compatible with the fluoride solution to be fed;
5. Solution feeders shall be of the positive displacement type and constructed of material compatible with the fluoride solution being fed;
6. The weight of the daily amount of fluoride fed to water shall be accurately determined;
7. Feeders shall be provided with anti-siphon valves on the discharge side. Wherever possible, positive anti-siphon breakers other than valves shall be provided;
8. A "day tank" capable of holding a 24-hour supply of solution shall be provided for systems not equipped with on-line continuous fluoride monitors;
9. All equipment shall be sized such that it will be operated in the 20 to 80 percent range of the scale, and are capable of feeding over the entire pumpage range of the plant;
10. Alarm signals are recommended to detect faulty operation of equipment; and
11. The fluoride solution should be added to the water supply at a point where any treatment processes following the point of addition would not remove the fluoride and where it will be mixed with the water. It is undesirable to inject the fluoride compound or solution directly on-line unless there are provisions for adequate mixing.

1.5.4.3 Metering

Metering of the total water to be fluoridated shall be provided, and the operation of the feeding equipment is to be controlled unless specifically exempted by AENV. Control of the feed rate shall be:

1. Automatic/ proportional controlled, whereby the fluoride feed rate is automatically adjusted in accordance with the flow changes to provide a constant pre-established dosage for all rates of flow; or
2. Automatic/ residual controlled, whereby a continuous automatic fluoride analyzer determines the residual fluoride level and adjusts the rate of feed accordingly; or

3. Compound loop controlled, whereby the feed rate is controlled by a flow proportional signal and residual analyzer signal to maintain a constant residual.

1.5.4.4 Alternate Compounds

Any one of the following fluoride compounds may be used:

1. Fluorosilicic acid;
2. Sodium fluoride; or,
3. Sodium silicofluoride.

Other fluoride compounds may be used if approved by AENV.

1.5.4.5 Chemical Storage and Ventilation

The fluoride chemicals storage area shall be marked "FLUORIDE CHEMICALS ONLY". The storage area should be in close proximity to the feeder, kept relatively dry, and provided with pallets, if using bagged chemical, to allow circulation of air and to keep the containers off the floor.

1.5.4.6 Record of Performance Monitoring

Accurate daily records shall be kept. These records shall include:

1. the daily reading of the water meter, which controls the fluoridation equipment or that which determines the amount of water to which the fluoride is added;

2. the daily volume of water fluoridated;

3. the daily weight of fluoride compound in the feeder;

4. the daily weight of fluoride compound in stock;

4. the daily weight of the fluoride compound fed to the water; and

5. the fluoride content of the raw and fluoridated water determined by laboratory analysis, with the frequency of measurement as follows:

   a. treated water being analyzed continuously or once daily, and

   b. raw water being analyzed at least once a week.

1.5.4.7 Sampling

In keeping the fluoride records outlined in Section 1.5.4.6, the following sampling procedures are required:

1. A sample of raw water and a sample of treated water shall be forwarded to an approved independent laboratory for fluoride analysis once a month;
2. On new installations or during start-ups of existing installations, weekly samples of raw and treated water for a period of not less than four consecutive weeks (or for a period as required by AENV) shall be submitted to either AENV or other designated laboratory to determine the fluoride concentration; and

3. In addition to the reports required, AENV may require other information that is deemed necessary.

1.5.4.8 Safety

The following safety procedures shall be maintained:

1. All equipment shall be maintained at a high standard of efficiency, and all areas and appliances shall be kept clean and free of dust. Wet or damp cleaning methods shall be employed wherever practicable;
2. Personal protective equipment shall be used during the clean-up, and appropriate covers shall be maintained over all fluoride solutions;
3. At all installations, safety features are to be considered and the necessary controls built into the installation to prevent an overdose of fluoride in the water. This shall be done either by use of day tanks or containers, anti-siphon devices, over-riding flow switches, sizing of pump and feeders, determining the length and duration of impulses, or other similar safety devices as approved by AENV;
4. Safety features shall also be provided to prevent spills and overflows as determined by AENV;
5. Individual dust respirators, chemical safety face shields, rubber gloves, and protective clothing shall be worn by all personnel when handling or being exposed to the fluoride dust;
6. Chemical respirators, rubber gloves, boots, chemical safety goggles and acid proof aprons shall be worn where acids are handled;
7. After use, all equipment shall be thoroughly cleaned and stored in an area free of fluoride dusts. Rubber articles shall be washed in water, and hands shall be washed after the equipment is stored; and
8. All protective devices, whether for routine or emergency use, shall be inspected periodically and maintained in good operating condition.

1.5.4.9 Repair and Maintenance

Upon notifying AENV and the appropriate Regional Health Authority, a fluoridation program may be discontinued permanently or temporarily when necessary to repair or replace equipment. If discontinued temporarily, fluoridation shall commence immediately after the repair or replacement is complete. Records shall be maintained and submitted during the period that the equipment is not in operation.

1.5.5 Health and Safety Act

The design and construction of all components of the waterworks system shall conform to the safety provisions of the Alberta Occupational Health and Safety Act and Regulations.

1.6 Waterworks Systems Operation Standards

1.6.1 System Operation Program

The proper operation and maintenance of waterworks system is essential to ensure ongoing sustained production and delivery of the best quality drinking water that is both wholesome and protective of public health. Thus, the system owner / operator shall develop an operation program to include routine operational procedures, monitoring and analytical procedures, emergency response planning, corrective action measures, cross-connection controls, etc. to ensure a reliable and well operated waterworks system. The operations program shall contain, at a minimum, all of the information in Table 1.4.

1.6.2 Reliability

The waterworks system shall provide an adequate quantity of safe drinking water in a reliable manner at all times. In determining whether a proposed public water system or an expansion or modification of an existing system is capable of providing an adequate quantity of water, the owner shall consider the immediate as well as the reasonably anticipated future needs of the system's consumers.

The owner shall prepare a Bacteriological Quality Monitoring Plan to include at a minimum, a system map or diagram showing the locations of:

1. water source;
2. storage, treatment and pressure regulation facilities;
3. distribution system;
4. pressure zones; and
5. sample collection sites.

The plan should be revised or expanded at any time the plan no longer ensures representative monitoring of the system. The Bacteriological Monitoring Plan should be made available to AENV for inspection upon request.

The owner shall ensure that the system is operated and maintained properly, and has appropriate backup facilities to protect against failures of the power supply, treatment process, equipment, or structure. Security measures shall address the safety of water source, water treatment processes, water storage facilities and the distribution system.

Water pressure at the customer's property line shall be maintained at the approved design pressure under maximum hourly design flow conditions. The minimum distribution pressure during peak demand design flow shall be 150 kPa.

1.6.3 Operations

1. The waterworks system shall be managed and operated in accordance with the AENV approval or the appropriate code of practice for the facility. The facility shall meet the minimum treatment performance requirements as outlined in Section 1.3, and the treated water shall meet the water quality as outlined in Section 1.1.
2. The plant shall be operated within its design capacity to supply treated water.

3. The owner shall not establish nor maintain a by-pass to divert water around any feature of a treatment process unless the Regional Director has approved the by-pass.
4. The owner shall not operate the plant when critical treatment equipment is inoperable.
5. The owner shall take preventative or corrective action when results of an inspection conducted by the Regional Director or monitoring reports indicate conditions, which are currently or may become a detriment to system operations.
6. The owner shall endeavour to protect waterworks systems from contamination due to cross-connections.
7. The owner shall develop and implement a cross-connection control program. The scope and complexity of the program should be directly related to the size of the system and the potential public health risk.

When an existing cross-connection poses a potential health or system hazard, the owner shall shut off water service until the cross-connection has been eliminated or controlled by the installation of a proper backflow prevention assembly.

Table 1.4
Operations Program Contents

1. Routine Operational Procedures, which shall, at a minimum, include:
    a. contact name and telephone numbers for the system owner, system operator, engineering consultants and equipment suppliers,
    b. operating instructions:
        i. general description of treatment process and operating procedures;
        i. performance requirements; and
        ii. location of equipment major controls
    c. general maintenance schedule; and
    d. general maintenance instructions for:
        i. treatment / process equipment;
        ii. monitoring equipment; and
        iii. pumping equipment; and
    e. the schedule and procedures for cleaning and flushing of the water distribution system, including potable water storage reservoirs.
2. Routine Operational Procedures for Monitoring and Analysis, which shall, at a minimum, include:
    a. operational and compliance tests to be performed,
    b. bacteriological quality monitoring plan,
    c. methods used for monitoring and analysis,
    d. locations of monitoring points; and
    e. laboratory data quality assurance information.
3. Emergency Response Plan, which shall, at a minimum, include steps to be taken in the event of the following:
    a. bacteriological results exceeding the prescribed limits,
    b. turbidity / particle counts exceeding the limits,
    c. chemical overfeed,
    d. no chemical or coagulant feed,
    e. low chlorine residual,
    f. equipment breakdown,
    g. flood,
    h. water distribution system pipeline break and repair, and the return of the pipeline to service,
    i. power failure,
    j. the waterworks system becoming inoperable, including steps in providing an alternate potable water supply, and
    k. list of contacts; Alberta Environment, Alberta Health, Regional Health Authorities, Fire Department, Disaster Coordinator, and other agencies.
4. Date of last update.

1.7 Potable Water Treatment Chemical standards

1.7.1 Use, Storage and Handling

The use, storage and handling of any hazardous chemicals at the water treatment plant shall be in accordance with the federal and provincial legislation for Workplace Hazardous Materials Information System (WHMIS).

1.7.2 Direct and Indirect Additives

The American National Standards Institute and National Sanitation Foundation Standards (ANSI/NSF) 60 and 61 shall be used to control potential adverse human health effects from products in contact with or added to water directly for treatment or indirectly during treatment, storage or transmission as follows:

1.7.2.1 Indirect Additives

All substances, materials or compounds (e.g. pipes, coatings, filter media, solders, valves, gaskets, lubricants, resins, process equipment, etc.) that may come in contact with water in the waterworks being treated to be potable and water that is potable shall conform to ANSI/NSF Standard 61 for health effects and the product certified for potable use by an agency accredited by the Standards Council of Canada, e.g. NSF, CSA, UL, etc.

The following exceptions apply:

- Any materials listed in the current NSF Standard 61 "Annex C".
- Existing waterworks (unless otherwise notified by the Regional Director).
- Portland cement based concrete. However, the Portland cement, any admixtures used in the concrete and concrete coatings shall be certified.
- If NSF certification does not exist for any substance, material or compound, the Regional Director, at his discretion, may approve formal food grade certification by a recognized agency (Health Canada, FDA, etc.).

1.7.2.2 Direct Additives

All substances, materials or compounds (e.g. coagulants, disinfectants, polymers, fluoride, ammonia, phosphates, caustic soda, etc.) that are added to water in the waterworks being treated to be potable and water that is potable shall conform to ANSI/NSF Standard 60 for health effects and the product certified for potable use by an agency accredited by the Standards Council of Canada, e.g. NSF, CSA, UL, etc.

The following exceptions apply:

- Materials, which are insoluble and non-reactive with the water or with other materials in the water and which are fully removed as part of the process (e.g. ballast sand, etc.). These materials shall be considered as materials that come in contact with the water.

Note: AWWA standards are recommended for setting specifications and checking product quality.

1.8 Potable Water Treatment Plant Waste – Handling and Disposal Standards

Provision shall be made for adequate treatment and/or disposal of all water treatment plant wastes. These include sanitary wastes, filter backwash, filter-to-waste and sludges.

1.8.1 Sanitary Wastes

All sanitary wastes from water treatment plants shall be handled by direct discharge to a sanitary sewer system or to an approved or registered wastewater treatment facility. No wastewater lines shall pass through potable water reservoirs.

1.8.2 Filter Backwash

Backwash wastewater may be discharged directly to a sanitary sewer system, if the sewers and the wastewater treatment plant can withstand the hydraulic surges.

Backwash wastewater is not to be discharged directly to an open body of water, unless it can be demonstrated that there are no significant adverse effects on the receiving body of water. Based on the quantity and quality of backwash waste and the sensitivity of the receiving body of water, AENV may request for an impact assessment study to ascertain the need for backwash waste treatment before discharging to the environment.

Recycle of filter backwash water will be permitted for a conventional system if the filter backwash water receives off-line treatment, or is equalized to minimize turbidity spikes and restrict the recycle flow to less than 10% of the raw water flow. Equalized filter backwash water shall be returned to a location upstream of the coagulant dosage point, so that all processes of a conventional plant are employed. Direct recycle of filter backwash water to a location upstream of the coagulant dosage point will be permitted providing that the conventional process is designed to accommodate the variations in raw water feed quality and flow rates that will occur. Recycling of filter backwash water will not be permitted for a direct-filtration system unless it receives off-line treatment.

1.8.3 Filter-to-Waste

Filter-to-waste may be discharged directly to a sanitary sewer system, if the sewers and the wastewater treatment plant can withstand the hydraulic surges. Filter-to-waste may also be recycled back into the head works without further treatment, or recycled back immediately upstream of the filters if the flow does not exceed 10% of the total inflow into the filters.

1.8.4 Sludges

Sludges generated at water treatment plants shall be treated and handled in a manner approved by AENV. The following sections deal with the various coagulant and softening sludges, and also outline the alternative methods of treatment and disposal.

1.8.4.1 Aluminum Sludge

Approval for the disposal of aluminium sludge shall be obtained on a site-specific basis. The following methods of handling and disposing of aluminum sludge may be used:

1. Direct discharge to a wastewater treatment plant or sanitary sewer. Consideration should be given to the potential beneficial and adverse effects on the wastewater treatment facility.
2. Lagooning. Lagoons can be used as permanent storage facilities, long-detention settling lagoons to provide freeze/thaw cycle with supernatant disposal, or drying beds using evaporation.
3. Mechanical thickening and dewatering. Once thickened and dewatered to approximately 20% solids, sludge can be placed at an approved disposal site, usually a landfill site used exclusively for sludge.
4. Direct discharge to a stream. This option will be approved only where there is a negligible environmental impact and it has been demonstrated that the aesthetics and downstream water users will not be affected. This option should only be considered if alternate management options are unavailable.
5. Land disposal. Land disposal to a sanitary landfill site or agricultural land of dilute or thickened and dewatered sludge is potentially harmful and shall be thoroughly reviewed with the approving authority prior to implementation.
6. Alum recovery. Alum recovery and re-use at water treatment plants is not considered a viable option; concerns with recovery costs and recycle of organics and heavy metals are the major reasons for rejection of this sludge handling alternative. It may, however, be feasible to recover water treatment plant alum sludge for re-use at a wastewater treatment plant utilizing chemical precipitation of phosphorus.
7. Reduction of sludge quantity. A reduction in the quantity of solids is possible by utilizing a number of management practices including pre-sedimentation by raw water storage, the use of polymer, or the effective design and operation of the coagulation/flocculation facilities.

1.8.4.2 Lime Sludge

Handling and disposal methods are similar to those for alum sludge (see Section 1.8.4.1), however under no circumstances may lime sludge be discharged directly to any watercourse in Alberta. Application shall be made to AENV for disposal of lime sludge on a case specific basis.

Because of potential deposition problems, the practice of lime sludge disposal to sewers is not recommended.

1.8.4.3 Sludges from Other Coagulants

There are a number of coagulants (such as ferric/ferrous compounds, polymers, polyelectrolytes, sodium aluminate), which are an alternative to aluminium-based coagulants. The handling and disposal of these other chemical sludges shall be approved by AENV on a site-specific basis.

1.9 Potable Water Transmission and Distribution Main Standards

1.9.1 Pipe Performance Standards

Where pipe performance standards exist, all materials that are used in the construction of the plant, transmission and distribution systems shall meet or exceed AWWA and/or CSA standards.

1.9.2 Frost Protection for Mains and Reservoirs

1.9.2.1 Mains

To prevent freezing and damage due to frost, pipes shall have a minimum cover above the crown of the pipe of:

1. 2.5 m, or
2. the depth of frost penetration for the location based on the coldest three years during the past 30 years, or, where this period of record is not available, the coldest year during the past 10 years with an appropriate safety factor.

Where these minimum frost protective covers cannot be achieved, AENV may allow an exemption if the owner can demonstrate incorporation of appropriate special precautions in the selection of pipe, bedding and insulation material.

1.9.2.2 Reservoirs

All treated water reservoirs, holding tanks, or storage water facilities shall have suitable watertight roofs to prevent reintroduction of contaminants that the treatment plant was designed to remove. A suitable cover shall be provided for access into the reservoir. Covers shall be made be watertight, and constructed so as to provide drainage away from the cover and prevent entrance of contamination into the stored water.

Reservoirs and appurtenances (such as overflows and vents) shall be designed to prevent reservoir contamination and damage from freezing. Elevated tanks and standpipes shall be insulated and hot water re-circulated, or heat traced, to prevent problems associated with ice formation.

1.9.3 Cross-Connection Controls

There shall be no physical connection between any waterworks systems and a sanitary or storm sewer that may allow the passage of wastewater into the potable water supply. Further, to prevent potential contamination, and avoid re-growths, no cooling water shall be returned into the potable water system.

The following cross-connection controls are necessary to preclude the entrance of contaminants into the water distribution system.

1.9.3.1 Horizontal Separation of Water Mains and Sewers

A watermain is defined as a pipeline that conveys water and forms an integral part of the water distribution system as defined in EPEA.

Unless otherwise approved by AENV, the minimum horizontal separation between a watermain and a storm or a sanitary sewer or manhole shall be 2.5 m, the distance being measured centre to centre.

Unusual conditions including excessive rock, dewatering problems, or congestion with other utilities may prevent the normal required horizontal separation of 2.5 m. Under these condition(s), AENV may approve a lesser separation distance, provided that the crown of the sewer pipe is at least 0.5 m below the watermain invert.

Where extreme conditions prevent the 2.5 m separation and vertical separation cannot be obtained, the sewer shall be constructed of pipe and joint materials, which are equivalent to watermain standards.

Under no circumstances shall the horizontal separation be less than 1.0 m.

1.9.3.2 Pipe Crossings

Under normal conditions, water mains shall cross above sewers with a sufficient vertical separation to allow for proper bedding and structural support of the water and sewer mains.

Where it is necessary for the watermain to cross below the sewer, the watermain shall be protected by providing:

1. a vertical separation of at least 0.5 m from watermain crown to sewer invert;
2. structural support of the sewer to prevent excessive joint deflection and settling; and
3. centering of the length of watermain at the point of crossing so that the joints are equidistant from the sewer.

1.9.3.3 Valve, Air Relief, Meter and Blow-Off Chambers

Chambers or pits containing valves, blow-offs, meters, or other water distribution appurtenances shall not be directly connected to a storm or sanitary sewer, nor shall blow-offs or air relief valves be directly connected to any sewer.

1.9.3.4 Backflow Prevention and Control

Backflow preventers shall be installed at any location where a connection is made to an approved waterworks system for the purpose of serving a hamlet, municipal development, privately owned development or a truck fill station located outside the service boundary of the approved waterworks system. Backflow preventers shall be installed in accordance with the latest edition of the Cross Connection Control Manual, published by AWWA (Western Canada Section).

1.9.4 Disinfection of Mains and Reservoirs

1.9.4.1 Mains

All new water mains shall be disinfected and flushed before being put into service in accordance with the latest edition of AWWA Standard C651 for Disinfecting Water Mains.

For existing water mains that are repaired, the line must be flushed until chlorine residual and turbidity levels are within normal operating ranges (average turbidity < 2.0 NTU and total chlorine residual > 0.1 mg/L), before putting the main back into service. Samples must be collected at the same time to determine the bacteriological quality of the water. The existing main may be returned to service prior to the completion of bacteriological testing in order to minimize the time customers are without water.

1.9.4.2 Reservoirs

Treated water storage reservoirs shall also be disinfected and flushed before being put into service, in accordance with the latest edition of AWWA Standard C652 for Disinfecting Reservoir.

1.9.4.3 Discharge of Superchlorinated Water

Chlorinated water used for disinfection of mains and reservoirs shall not be directly drained into the storm sewer or into an open body of water; dechlorination is required before being discharged into the environment.

Super chlorinated water may be discharged into a wastewater system, if the sewer by-law of the municipality allows this discharge.

1.9.5 Layout

Water distribution systems shall be designed to eliminate dead-end sections, whenever possible. In cases where newly constructed dead-end mains are unavoidable, flushing devices shall be installed to prevent stagnation and to facilitate return to service procedures following repairs. These devices may include hydrants, blow-off valves, stand pipes equipped with gate valves or other devices designed to adequately flush dead-end mains.

1.10 Potable Water Quality Performance Monitoring Standards

1.10.1 General

Establishing reasonable and appropriate monitoring requirements for waterworks facilities is a key factor in ensuring safe drinking water. Alberta Environment considers monitoring to fall into one of the following general categories:

1. operational monitoring (See section 2.3.8);
2. treatment performance and compliance monitoring; and
3. follow-up or issue oriented monitoring.

"Operational monitoring" consists of the sampling regime for proper operation of the waterworks system. The use of these parameters by the system operator to monitor operation is discretionary.

"Treatment Performance and Compliance monitoring" and "Follow-up or Issue Oriented monitoring" consists of the sampling regime that is required to ensure ongoing sustained production and delivery of high quality of drinking water. These are minimum mandatory requirements, and shall be complied with.

1.10.2 Analytical Requirements

1. With respect to any monitoring required pursuant to this Section, all samples shall be collected, preserved, stored, handled; and analyzed in accordance with:

    a. the *Standard Methods for the Examination of Water and Wastewater*, published by the American Public Health Association, the American Waterworks Association and the Water Environment Federation, as amended or replaced from time to time; or

    b. a method authorized in writing by the Regional Director.

2. With respect to monitoring of cysts and oocysts, all samples shall be collected, preserved, stored, handled; and analyzed in accordance with the USEPA Methodology 1623.
3. Any analysis of a sample required pursuant to this Section, shall be done only in an approved laboratory, unless otherwise specified in writing by the Regional Director.
4. Only the Alberta Provincial Laboratory shall conduct any analysis of a sample for bacteriological quality required pursuant to this Section for Public Health, unless otherwise specified in writing by the Regional Director.
5. Where on-line instruments are specified or allowed, such instruments shall be kept maintained and calibrated in accordance with the manufacturer's recommendations.

1.10.3 Compliance Monitoring

This section outlines the specific parameters that have to be monitored to ensure that water is safe, including the sampling locations and the monitoring frequencies.

1.10.3.1 Bacteriological

All systems shall comply with the following with respect to meeting monitoring frequency and bacteriological quality in potable water:

1. Sampling Location

    Bacteriological samples shall be collected from representative points after treatment and throughout the distribution system after the first service connection.

2. Monitoring Frequency

    The number of required routine samples shall be in accordance with the GCDWQ, or as directed by AENV. The sampling shall be evenly distributed through the sampling period.

3. Invalid Samples

When a bacteriological sample is determined invalid by the laboratory, the owner shall:

a. not include the sample in the required number of samples; and

b. collect and submit for analysis, an additional drinking water sample from the same location as each invalid sample within twenty-four hours of notification by the laboratory or AENV.

4. Compliance Criteria

Compliance criteria for bacteriological quality shall be in accordance with GCDWQ.

1.10.3.2 Physical Parameters, Organic & Inorganic Chemicals and Pesticides

1. Parameters to be monitored

A complete analysis shall consist of the primary and secondary substances and should include all physical parameters, organic and inorganic chemicals and pesticides. The primary substances are those substances with MACs in the GCDWQ and which are known to cause adverse effects on health. The secondary substances are those substances with AOs in the GCDWQ with limits below those considered to constitute no health hazard and the parameters with Operational Guidance Value, and some of the parameters without guidelines identified in the GCDWQ:

a. Physical parameters (Primary and Secondary)

colour, pH, total dissolved solids, turbidity and UV absorbance (not in the GCDWQ);

b. Inorganic chemicals (Primary)

antimony, arsenic, barium, boron, bromate, cadmium, chloramines, chromium, cyanide, fluoride, lead, mercury, nitrate and nitrite, selenium, and uranium;

c. Inorganic and Organic Chemicals (Secondary)

aluminum, ammonia, calcium, chloride, copper, hardness, iron, magnesium, manganese, silver, sodium, sulphate, sulphide, total organic carbon, xylenes (total) and zinc

d. Organic Chemicals and Pesticides (Primary)

Atrazine +metabolites, benzene, benzo(a)pyrene, bromoxynil, carbon tetrachloride, chlorpyrifos, cyanazine, cyanobacterial toxins (as microcystin – LR – for surface water systems only), diazinon, dicamba, dichlorobenzene 1, 2-, dichlorobenzene 1.4-, dichlorethane 1, 2-, dichloromethane, 2, 4- dichlorophenol, 2, 4-D, diclofop-methyl, diuron, dimethoate, ethylbenzene, glyphosate, malathion, methoxychlor, metolachlor, metribuzin, monochlorobenzene, nitrilotriacetic acid (NTA), pentachlorophenol, picloram, simazine, terbufos, tetrachloroethylene, tetrachlorophenol 2, 3, 4, 6-, toluene, triallate, trichloroethylene, trichlorophenol 2, 4, 6-, trifluralin, vinyl chloride.

For specific systems, the Regional Director, at his or her discretion, may revise the list of primary and secondary substances to be monitored.

2. Sampling Location

   a. The samples shall be collected from a point representative of each source, after treatment, and prior to entry to the distribution system. The point of collection shall be designated as "Sampling Location" and confirmed by AENV.

   b. For multiple sources or well fields within a single system, in which water is blended prior to entry into the distribution system, the samples should be collected after treatment, within the water distribution system, at a location where the water from all well fields has been blended.

      Each sample shall be taken at the same point unless conditions make another sampling point more representative of the water produced by the treatment plant.

   c. For multiple sources or well fields within a single system, in which water is not blended prior to entry into the distribution system, the samples should be collected after treatment at least in one of the wells, at the point of entry into the water distribution system.

   d. When treatment is provided for one or more contaminants, AENV would require sampling before treatment for the affected parameter. The "Source Sampling Location" for raw water supply should be confirmed by AENV. For groundwater supply requiring treatment, each well should be sampled at the source.

Note: Lead and copper sampling shall be done within the distribution system on a flushed cold-water sample.

3. Monitoring Frequency

   a. The frequency of monitoring conducted to determine compliance with the MACs and AOs for the primary and secondary substances shall be once per year for groundwater supplies and twice per year, once in the summer and once in the winter, for surface water supplies at each designated "Sampling Location". For groundwater requiring treatment, one additional sample shall be required at each "Source Sampling Location."

   b. For high quality groundwater systems, where the results of sampling indicate that AOs and MACs have not exceeded, the frequency of monitoring may be reduced to once every three years for physical and inorganic chemical parameters, and once every five years for organic chemical and pesticide parameters.

   c. For waterworks systems consisting solely of a water distribution system, no monitoring is required for organic chemicals and pesticides. Physical and inorganic chemical parameters shall be monitored once every three years.

   d. Where the results of sampling indicate that MACs have been exceeded, AENV will require that one additional confirmation sample be collected as soon as possible (but not to exceed two weeks) after the initial sample results are received. Systems that exceed the MACs in confirmation samples shall monitor quarterly beginning in the next quarter after the violation occurred, or as directed by AENV. The owner may revert back to the frequencies specified in sub-section 1.10.3.2(3)(a) above, provided that the system is reliably and consistently producing water below the MACs. AENV will make this determination based on a minimum of two quarterly samples for groundwater systems, and a minimum of four quarterly samples for surface water systems.

e. For high quality water well fields within a single system, in which water is not blended prior to entry into the distribution system, at least one well shall be monitored each year, and rotated so that all wells have been monitored within the previous three years.

4. Compliance

a. For systems that are monitoring once a year, the system is out of compliance, if the level of a substance at any sampling point is greater than the MAC. If a confirmation sample is required by AENV, the determination of compliance will be based on the average of the two samples.

b. For systems that are monitoring at a frequency greater than once a year, compliance with the MACs is determined by a running annual average at any sampling point. If the average at any sampling point is greater than the MAC, then the system is out of compliance. Any sample below the detection limit shall be calculated at zero for the purpose of determining the annual average.

1.10.3.3 Turbidity

All systems requiring turbidity reduction shall comply with the following:

1. Source Water Turbidity Monitoring

Source water turbidity shall be monitored at least once per day on a representative sample collected before the addition of any chemicals.

2. Treated Water Turbidity Monitoring

For rapid sand filtration, slow sand filtration, membrane filtration and cartridge filtration systems, treated water turbidity shall be monitored at all individual filters / filter modules upstream of the clear water tank, and at the combined filter effluent line. The measurements shall be made continuously at no more than five-minute intervals with an on-line turbidimeter.

3. Compliance

System compliance for turbidity shall be in accordance with the Performance Standards detailed in Section 1.3.1.

1.10.3.4 Fluoride

All systems practicing fluoridation shall comply with the performance monitoring and performance requirements detailed in Section 1.5.4.6 and Section 1.3.3 respectively.

1.10.3.5 Iron and Manganese

All systems practicing iron and manganese reduction shall comply with the following:

1. Source Water Monitoring

   Source water iron and manganese shall be monitored at least once per week, at a location prior to any chemical addition or treatment unit.

2. Treated Water Iron and Manganese Monitoring

   Treated water iron and manganese shall be monitored after treatment, at the point of entry into the water distribution system. If monitored by taking grab samples, the frequency of sampling shall be based on one sample per day, five days per week, except on statutory holiday if it falls on a monitoring day within that week. If monitored continuously, one sample should be taken every five minutes with an on-line meter.

3. Compliance

   Compliance criteria for iron and manganese, where iron and manganese removal are practiced, shall be 0.3 mg/L and 0.08 mg/L respectively.

1.10.3.6 Trihalomethanes and Bromodichloromethane

1. Sampling Location and Monitoring Frequency

   a. Surface Water Systems serving a population greater than 10,000 people

      Surface water systems, providing water treated with chlorine shall monitor as follows:

      i. Collect four samples per treatment plant every month. For systems that exceed the MAC, increased frequency may be required during peak by-product formation periods. The samples should be taken within a twenty-four hour period. One of the samples should be taken at the water treatment plant, one from the extreme end of the distribution system and the other two samples from representative locations in the distribution system. The samples should be analyzed for total trihalomethanes (TTHM) and bromodichloromethane (BDCM).

      ii. If the TTHM results from the same location (based on running annual average from the previous 12 months), and the BDCM results (based on any single result) are less than the respective MAC, subsequent monitoring shall be conducted, at a minimum, in the following manner:

         A. Collect four samples per treatment plant every three months, unless otherwise authorized in writing by the Regional Director. The samples should be taken within a twenty-four hour period. One of the samples should be taken at the water treatment plant, one from the extreme end of the distribution system and the other two samples from representative locations in the distribution system. The samples should be analyzed for TTHM and BDCM.

b. Surface Water Systems serving a population less than 10,000 people

Surface water systems, providing water treated with chlorine shall monitor as follows:

i. Collect four samples per treatment plant every three months. The samples should be taken within a twenty-four hour period. One of the samples should be taken at the water treatment plant, one from the extreme end of the distribution system and the other two samples from representative locations in the distribution system. The samples should be analyzed for total TTHM and BDCM.

ii. If the TTHM results (based on running annual average from the previous 12 months) and the BDCM results (based on any single result) are less than the respective MAC, subsequent monitoring shall be conducted, at a minimum, in the following manner:

A. Collect one sample per treatment plant every three months from the extreme end of the distribution system, unless otherwise authorized in writing by the Regional Director. The sample should be analyzed for total TTHM and BDCM.

c. High Quality Groundwater Systems

i. New and existing systems with no TTHM or BDCM data, and with total organic carbon (TOC) > 2 mg/L in the raw water:

These systems shall monitor every quarter at the furthest point in the distribution system for one year to establish seasonal variations. If no sample exceeds 100 ug/L for TTHMs and 16 ug/L for BDCM, then the frequency of monitoring may be reduced to once every three years or as determined by the Regional Director.

ii. New and existing systems with no TTHM or BDCM data, and with TOC < 2 mg/L in the raw water:

These systems shall monitor twice a year, once in the summer and once in the winter, at the furthest point in the distribution system to establish seasonal variations. If no sample exceeds 100 ug/L for TTHMs and 16 ug/L for BDCM, then the frequency of monitoring may be reduced to once every three years or as determined by the Regional Director.

iii. Existing Systems with historical data and TOC < 2 mg/L in the raw water:

If there are samples collected under current disinfection practices:

A. covering two or more seasons (summer/winter) and indicate less than 100 ug/L TTHM and less than 16 BDCM, would require sampling once every three years at the furthest point in the distribution system, or as determined by the Regional Director,

B. covering only one season (summer/winter) and indicate less than 100 ug/L TTHM and less than 16 ug/l BDCM, would require additional sampling to identify seasonal variation (summer/winter). If none of the samples exceed TTHM concentration of 100 ug/L, or BDCM concentration of 16 ug/L, then the frequency of monitoring may be reduced to once every three years or as determined by the Regional Director, and

C. that indicate more than 100 ug/L of TTHM or more than 16 ug/L of BDCM, would require standard frequency of testing of four times per year at the furthest point in the distribution system and reassessment after one year.

iv. Existing Systems with historical data and TOC > 2 mg/L in raw water:

If there are samples collected under current disinfection practices:

A. covering all four seasons and indicate less than 100 ug/L TTHM and less than 16 ug/l BDCM, would require sampling at the furthest point in the distribution system once every three years, or as determined by the Regional Director,

B. covering less than four seasons and indicate less than 100 ug/L TTHM and less than 16 ug/l BDCM, would require additional sampling at the furthest point in the distribution system to establish seasonal variations. If none of the samples exceed the TTHM and BDCM limits, then the sampling may be reduced to once every three years or as determined by the Regional Director, and

C. that indicate more than 100 ug/L of TTHM or more than 16 ug/L of BDCM in any sample, would require standard frequency of testing of 4 times per year at the furthest point in the distribution system and reassessment after one year.

v. Where the disinfection process is altered resulting in increased application of the disinfectant, monitoring shall commence as outlined in Sections i and ii, as a new system.

d. Systems consisting solely of distribution system

i. Where water is obtained from a regional system and no re-chlorination or other disinfectant is added, then the monitoring results available from the regional system and from their distribution system may be used for determining the frequency of monitoring required. Based upon the available information from the regional supplier the TTHM and BDCM monitoring frequency shall be as follows:

Where there are samples:

A. covering two or more seasons (summer/winter) and indicate less than 100 ug/L TTHM and less than 16 ug/l BDCM, would require sampling once every three years, or as determined by the Regional Director,

B. covering only one season (summer/winter) and indicate less than 100 ug/L TTHM and less than 16 ug/l BDCM, would require additional sampling to identify seasonal variation. If none of the samples exceed TTHM concentration of 100 ug/L, or BDCM concentration of 16 ug/L, then the frequency of monitoring may be reduced to once every three years or as determined by the Regional Director, and

C. that indicate more than 100 ug/L of TTHM or more than 16 ug/L of BDCM require standard frequency of testing of 4 times per year and reassessment after one year.

Note: The Regional Director, at his discretion, may exempt or reduce the frequency of monitoring for communities on a regional system, if TTHM and BDCM monitored downstream of these communities are below the respective MAC.

ii. Where water is obtained from a regional system, and re-chlorination or other disinfectants are added, the monitoring frequency shall be as follows:

A. samples shall be collected to cover every quarter and if no sample exceeds 100 ug/L for TTHMs and 16 ug/L for BDCM after one year, then the frequency of monitoring may be reduced to once every three years or as determined by the Regional Director, and

B. if the TTHM concentrations exceed 100 ug/L, or BDCM concentrations exceed 16 ug/L, then the system shall be continued to be monitored every quarter, and reassessed after one year.

2. Compliance

Compliance with the MAC for TTHM is determined by a running annual average of all samples taken at the same location during any twelve-month period. If the average exceeds the MAC, then the system is out of compliance.

Compliance with the MAC for BDCM is based on any single sample exceeding the limit.

If TTHM or BDCM result exceeds the applicable MAC, then the initial monitoring program shall be continued or recommenced.

1.10.3.7 Disinfection

1. Establishing the level of reduction for cysts, oocysts and viruses

i. The Regional Director will first determine the total level of cysts and oocysts reduction required through filtration and disinfection, based on the source water quality (Section 1.2.1).

ii. The Regional Director will then determine the allowable filtration credit based on the type and performance of filtration system. For systems not meeting the turbidity requirements outlined in Section 1.3.1, The Regional Director may grant reduced filtration credit or no filtration credit, as per Section 1.4.1.1.

iii. Based on sub-sections i and ii, the Regional Director will establish the level of cysts, oocysts and viruses reduction required through disinfection.

The Regional Director will periodically review the data, and would adjust, as necessary, the level of disinfection the owner should provide to protect the public health.

2. The owner shall calculate the total level of reduction of cysts, oocysts and viruses achieved, each day the system is in operation. The total level of reduction achieved will be based on:

   a. cysts, oocysts and virus reduction credit granted by the Regional Director for filtration, and

   b. the level of reduction of cysts, oocysts and viruses achieved through disinfection CT_{actual} or IT_{actual}.

Chlorine

1. Monitoring disinfectant concentration

   a. The residual disinfectant concentration “C” used for the calculation of “CT” shall be measured continuously at no more than five-minute intervals with an on-line analyser. “C” shall be measured at the same point where “T” is measured for the “CT” calculation, and

   b. the residual disinfectant concentration within the distribution system shall be measured:

      i. continuously after the clear water tank at the point of entry into the water distribution system; or

      ii. at least once daily, at representative points within the water distribution system; and

      iii. at the same time and at the same location as the bacteriological quality sample is collected.

   Note: For waterworks systems using high quality groundwater, and for waterworks systems consisting solely of a water distribution system, frequency of monitoring may be reduced to five days per week excepting statutory holidays.

2. Determination of disinfectant contact time (T_{10})

   a. T_{10} values can be significantly different from calculated detention times, T, (volume/flow) and shall be determined by tracer study or by reference to typical baffling conditions. Use of the T_{10} ensures that 90% of the water will be better treated than the minimum.

   b. T_{10} shall be calculated at maximum hourly flow.

c. For pipelines, T_{10} is calculated by dividing the internal volume of the pipe by the maximum hourly flow rate through that pipe.

d. For all other system components, tracer studies or empirical methods shall be used to determine T_{10}.

3. Tracer studies

a. Tracer studies shall be conducted on all system components for which similar contact times are not documented.

b. Three tracer studies shall be done for different flow conditions at various depths of clear-water tanks.

c. The tracer studies shall be conducted in accordance with good engineering practices using methods acceptable to AENV.

4. Empirical Methods

Empirical methods may be used to calculate T_{10}, if the owner can demonstrate that system components have configuration similar to components on which tracer studies have been conducted. See Appendix D for illustration of typical baffling conditions in reservoirs.

5. Determining the level of reduction for cysts, oocysts and viruses achieved

a. In order to determine the level of reduction of cysts, oocysts and viruses, the owner shall monitor the following daily:

   i. temperature of the disinfected water at each residual disinfectant concentration sampling point used for CT calculations;

   ii. if using chlorine, pH of the disinfected water at each chlorine residual disinfectant concentration sampling point used for CT calculations;

   iii. the filled capacity/depth of the clear water tank during maximum hourly flow (based on historical information), to determine T_{10}; and

   iv. the disinfectant concentration "C" of the water at the point for which "T" is calculated, as per sub-section (1) above.

b. The log reduction through disinfection shall be calculated each day the system is in operation. CT_{actual} shall be determined using the lowest recorded value for "C" and "T_{10}" at the maximum hourly flow. $CT_{required}$ shall be referenced from the Tables in Appendix A or Appendix B.

6. Compliance

The system will be considered in compliance with the requirement to inactivate cysts, oocysts and viruses when CT_{actual} due to disinfection, exceeds the $CT_{required}$ for that facility. CT_{actual} for cysts and oocysts may be less than $CT_{required}$ for cysts and oocysts for a maximum of one day a month; CT_{actual} for viruses shall exceed the $CT_{required}$ at all times.

Ultraviolet Light

If UV light is used as the primary disinfectant, to receive UV disinfection credit, utilities shall install and operate UV installations under the following conditions.

1. Utilities shall demonstrate a UV dose for the reactor, using the results of a UV validation test and ongoing monitoring, for the inactivation of cysts and oocysts and in accordance with the UV dose / inactivation Tables in Appendix C.
2. The UV reactor shall be validated by a full-scale bioassay by an independent third party against acceptable test organisms, with similar hydraulics and lamp/baffle layout as the utility installation.
3. The UV Validation Certificate shall be supplied by the manufacturer and shall list the acceptable valid operating range of the reactor with information on maximum/minimum flows, UV %T, and UV dose.
4. Doses for viruses are significantly higher than for protozoans, so UV is not a recommended technology for virus inactivation. UV light shall be followed by a secondary disinfectant such as chlorine to meet virus reduction requirements and to maintain a residual in the water distribution system.
5. Surface water and GWUDI systems with filtration shall meet the turbidity requirement of 0.3 NTU, for water entering the UV reactor, unless exempted by the Regional Director. For GWUDI systems without filtration, the turbidity of water entering the UV reactor shall not exceed 1 NTU, unless exempted by the Regional Director.
6. Provision should be made to shut down the plant or bypass the water to waste if the UV dose drops below the $IT_{required}$ (bypass capabilities are recommended). Under this circumstance, then no more than 1% of the water in a month, and no more than 2% of the water per day shall pass the UV units without the required level of treatment. The utility shall monitor daily the amount of time the reactor is off, or operating outside the validated performance range, while water is passing through it.
7. UV intensity sensor readings, flow through the reactors, and temperature and lamp status, shall be monitored continuously, average and minimum UV dose per reactor and per plant shall be calculated daily, and UV %T of the water entering the reactor shall be monitored at least once daily.
8. UV sensors shall be calibrated monthly against a reference sensor, and the reference sensor checked annually by the supplier or an independent third party. Data demonstrating stable sensor performance may be used, at the discretion of AENV, to decrease calibration frequencies.
9. Appropriate startup, shutdown, emergency and maintenance procedures shall be in place.
10. The log reduction through UV disinfection shall be calculated each day the system is in operation. IT_{actual} shall be the lowest recorded dosage for that day. $IT_{required}$ shall be the Reduction Equivalent Dose for the UV reactor achieved during the validation process.
11. The system will be considered in compliance with the requirement to inactivate cysts, oocysts and viruses when IT_{actual} due to disinfection, exceeds the $IT_{required}$ for that facility. IT_{actual} may be outside the validated performance dose as detailed in item 6 above.

1.10.4 Issue Oriented and Follow-up Monitoring

1.10.4.1 General

1. Follow-up action by the owner is required when the system does not meet the minimum potable water quality stipulated in Section 1.1 or the minimum performance requirements for treatment stipulated in Sections 1.3.
2. When a violation of MAC or minimum performance requirements for treatment occurs, the municipality shall:
    a. notify the Regional Director in accordance with Section 1.11,
    b. determine the cause of the contamination or operational problems, and
    c. take action as directed by the Regional Director.

1.10.4.2 Bacteriological

When bacteriological quality of the potable water fails, the follow-up monitoring, notification of the problem, corrective actions and public health intervention shall be in accordance with Alberta Environment's and Health's policy titled Communication and Action Protocol for Failed Bacteriological Results (including potential failures) in Drinking Water for Waterworks Systems Authorized under the Environmental Protection and Enhancement Act, as amended or replaced from time to time.

1.10.4.3 Disinfection

1. When CT_{actual} or IT_{actual} is less than the $CT_{required}$ or $IT_{required}$, the owner shall:
    a. stop water production until the CT_{actual} or IT_{actual} exceeds the $CT_{required}$ or $IT_{required}$,
    b. notify the Regional Director in accordance with section 1.11, and
    c. undertake corrective actions established in consultation with the Regional Director.
2. If the residual disinfectant concentration at the location where "C" is measured for the calculation of "CT", measured as free or combined chlorine, is less than 0.2 mg/L, the owner shall:
    a. take immediate actions (usually increasing disinfectant dosage and/or cleaning clear-water tank),
    b. increase the monitoring frequency until the residual meets the specified limit, and
    c. notify the Regional Director in accordance with section 1.11.
3. If chlorine residual entering or within the distribution system is less than 0.1 mg/L, the owner shall:
    a. take immediate actions (usually flushing and/or increasing disinfectant dosage) to obtain the residual, and

b. increase the monitoring frequency until the residual is detectable.

1.10.4.4 Fluoride

1. The fluoridation system shall be shut down if the owner is unable to test fluoride concentrations.
2. If the daily fluoride residual varies outside the approved range (typically 0.8 mg/L +/- 0.2 mg/L), the owner shall:
   a. resample and check calculated dosage,
   b. adjust and recalibrate the feed rate, and
   c. take a sample to verify that proper fluoride residual levels have been obtained.
3. If the fluoride residual levels exceed 1.5 mg/L or if the fluoride residual levels vary outside the approved range (typically 0.8 mg/L +/- 0.2 mg/L) on two consecutive days, the owner shall:
   a. resample and check calculated dosage,
   b. notify the Regional Director in accordance with Section 1.11, and
   c. undertake corrective actions established in consultation with the Regional Director.

   Note: The optimum concentration +/- 0.2 mg/L applies if the Regional Director approves the optimum concentration to be less than of 0.8 mg/L.

1.10.4.5 Turbidity

1. If individual filter treated water turbidity exceeds 0.3 NTU for more than 15 minutes per day, the owner shall:
   a. notify the Regional Director in accordance with Section 1.11, and
   b. undertake corrective actions established in consultation with the Regional Director.

1.10.4.6 Organic and Inorganic Chemicals, Pesticides, THM and BDCM

For organic and inorganic chemicals, pesticides, THMs and BDCM, follow-up monitoring shall be conducted in accordance with the procedures outlined in section 1.10.3.2 and 1.10.3.6.

1.11 Potable Water Quality Record Keeping and Reporting Standards

1.11.1 Record Keeping

All records shall bear the signature of the operator responsible for the waterworks system. Municipalities shall keep these records available for inspection by the Regional Director and send the records to the Regional Director if requested.

1.11.1.1 Five Year Records

The municipalities shall record the following information and maintain the following records for at least five years from the date the record was created:

1. Bacteriological analysis results;
2. Turbidity analysis results;
3. Daily records, including but not limited to:
    a. flow meter readings,
    b. chlorine concentrations,
    c. turbidity analysis results, and
    d. information on level of inactivation of Giardia cysts, Cryptosporidium oocysts and viruses achieved through disinfection:
        i. temperature at each residual concentration sampling point;
        ii. pH if using chlorine;
        iii. peak flow;
        iv. filled capacity/depth of clearwater tank;
        v. disinfectant contact time T, and corresponding concentration C; and
        vi. inactivation ratio;
    e. treatment chemical dosages,
    f. iron and manganese concentrations,
    g. all fluoridation information required under Section 1.5.4,
    h. all electronic and monthly reports submitted to AENV, and
    i. records of action taken by the municipality to correct contraventions of potable water quality limits (MAC or IMAC), including the following information for each contravention:
        i. name and address of the person who discovered the contravention; and
        ii. copies of all notifications to the public.

1.11.1.2 Lifetime Records

The municipality shall maintain the following records for the life of the waterworks system:

1. The system operations program;
2. copies of all:
    a. applications submitted to AENV for the approval or registration regarding the waterworks system and correspondence related to the approval or registration,
    b. engineering drawings and specifications,
    c. project reports,
    d. construction documents,
    e. record drawings,
    f. all reports of inspections conducted by AENV,

    g. all correspondence sent to AENV regarding a proposed extension of a water distribution system, replacement of a portion of a water distribution system, expansion or modification of potable water storage within the water distribution system,
    h. all approvals or registrations issued under the Act for the waterworks system,
    i. all annual reports, and
    j. all reports prepared pursuant to 1.11.2, and
3. all physical, organic and inorganic chemical and pesticide analytical results required pursuant to any approval or code, excluding daily monitoring.

1.11.1.3 Sampling and Analysis Records

The results and records in 1.11.1.1 and 1.11.1.2 (2) shall contain, at a minimum, all of the following information:

1. the date, location and time of monitoring, and the name of the person collecting the sample;
2. identification of the sample type, including, but not limited to whether the sample is a routine water distribution system sample, repeat sample, source or potable water sample, or other special purpose sample;
3. date of analysis;
4. laboratory name and person responsible for performing analysis;
5. the analytical method used; and
6. the results of the analysis.

1.11.2 Reporting Requirements

1.11.2.1 Contravention Reporting

1. The owner shall immediately report to the Regional Director any contravention of the Approval or the Code, either:
    a. by telephone at (780) 422-4505, or
    b. by a method:
        i. in compliance with the release reporting provisions in the Act and the regulations; or
        ii. authorized in writing by the Regional Director.
2. The owner shall immediately report to the Regional Director by a method under sub-section 1, any structural or equipment malfunction in the waterworks system that may affect the quality or supply of potable water.
3. In addition to the immediate report in sub-section 1, the municipality shall provide a report to the Regional Director:
    a. in writing, or
    b. by a method:

    i. in compliance with the release reporting provisions in the Act and the regulations; or
    ii. authorized in writing by the Regional Director within seven (7) calendar days after the discovery of the contravention, or within another time period specified in writing by the Regional Director, unless the requirement for the report is waived by the Regional Director.

4. The report required under sub-section 3 shall contain, at a minimum, the following information:
    a. a description of the contravention,
    b. the date of the contravention,
    c. the duration of the contravention,
    d. the legal land description of the location of the contravention,
    e. an explanation as to why the contravention occurred,
    f. a summary of all preventive measures and actions that were taken prior to the contravention,
    g. a summary of all measures and actions that were taken to mitigate any effects of the contravention,
    h. a summary of all measures that will be taken to address any remaining effects and potential effects related to the contravention,
    i. the number of the approval or registration issued under the Act for the waterworks system, and the name of the person who held the approval or registration at the time the contravention occurred,
    j. the name, address, work phone number and responsibilities of all persons operating the waterworks system at the time the contravention occurred,
    k. the name, address, work phone number and responsibilities of all persons who had charge, management or control of the waterworks system at the time that the contravention occurred,
    l. a summary of proposed measures that will prevent future contraventions, including a schedule of implementation for these measures,
    m. any information that was maintained or recorded under an Approval or a Code of Practice, as a result of the incident, and
    n. any other information required by the Regional Director in writing.

1.11.2.2 Monthly Reporting

The owner shall compile monthly reports.

1. The monthly report in shall include, at a minimum:
    a. the name and telephone number of all operators in direct charge,
    b. the analytical results for all parameters required to be monitored in accordance with an Approval or a Code of Practice during the month,

   c. the locations of all sampling performed during the month in accordance with an Approval or a Code of Practice,
   d. the name and manufacturer of all treatment chemicals added during the month, and each manufacturer as listed in the *Standard 60*, published by the American National Standards Institute and the National Sanitation Foundation (ANSI/NSF), as amended or replaced from time to time, and
   e. the results of all required measurements conducted during the month in accordance with an Approval or a Code of Practice.

1.11.2.3 Annual Reporting

1. In addition to any other reporting required under the Act, the regulations, an Approval or a Code of Practice, the owner shall submit to the Regional Director an annual report, by February 28 of the year following the calendar year in which the information on which the report is based was collected.
2. The annual report shall contain, at a minimum, all of the following information:
   a. a summary of the monthly reports, specifying the monthly minimum, average, and maximum results for each parameter monitored,
   b. the results of any other compliance monitoring done during the year pursuant to an Approval or a Code of Practice, that was not included in any monthly report, and
   c. a description of any problems experienced and corrective actions taken at the waterworks system during the year with respect to environmental matters.

1.11.2.4 Electronic Reporting

1. The Regional Director may, by notice in writing, require the owner to submit periodic reports:
   a. in an electronic format; and
   b. more frequently than specified in Sections1.11.2.2 and 1.11.2.3.
2. The registration holder who receives a notice as specified in sub-section 1 shall comply with the notice.

1.12 Laboratory Data Quality Assurance Standards

All analytical data submitted to the Department shall be analysed by laboratories accredited to the requirements of ISO/IEC 17025 - General requirements for the competence of testing and calibration laboratories, for the drinking water tests methods specified by the Regional Director. The exception to this requirement is the analysis done by municipalities in accordance with AENV's Alternate Laboratory Data Quality Assurance Program.

Accreditation to the laboratory shall be granted by an agency that meets the requirements of ISO 17011 - Conformity assessment - General requirements for accreditation bodies accrediting conformity assessment bodies, or its predecessor ISO Guide 58, General Criteria for the Operation and Mutual Recognition of Laboratory Accreditation Systems; and is a full member signatory to the International Laboratory Accreditation Cooperation.

1.13 Facility Risk Assessment Standards

Municipalities / system owners/operators shall undertake a risk assessment of the waterworks systems from source to tap to ascertain the integrity, reliability and the long term sustainability of the system to provide safe drinking water to the consumer. This independent assessment shall be done every five years by a third party approved by the Regional Director, using the criteria established in Section 2.8 – Facility Risk Assessment Guidelines, or as determined by the Regional Director. Based on this assessment, the facility shall be rated from low to high risk in supplying safe drinking water in accordance with the Risk Factor Tables 2.8, 2.9 and 2.10.

This report shall be submitted to the Regional Director for review and follow-up. An independent assessment of the facility is necessary to assure the general public and the Regional Director that municipalities / system owners/operators are committed to take the required action to reduce any potential risk in providing safe drinking water to the consumer.

1.14 Facility Classification and Operator Certification Standards

Alberta Environment classifies all waterworks facilities based on staff recommendations and review by the Alberta Operator Certification Advisory Committee. The system owner or authorized representative may also request a review of a facility classification. The classification of a water distribution system is based upon the population served by that facility, while the classification of a water treatment facility is based on the degree of difficulty of operating that facility. The facility classification shall be based on AENV's *Water and Wastewater Operators' Certification Guidelines,* as amended or replaced from time to time.

Specified waterworks systems in Alberta shall have certified operators to supervise and / or carry out day-to-day operation of the system. The level of operator certification is the same as the classification of the facility. The operator certification shall be based on AENV's *Water and Wastewater Operators' Certification Guidelines,* as amended or replaced from time to time.

2.0 Waterworks Systems Guidelines

2.1 Design Criteria

2.1.1 Water Quantity Requirements

A general correlation exists between the available quantity of drinking water and the level of public health within a community. The waterworks system shall be capable of providing sufficient quantities of water to meet the needs of consumers, meaning that the proposed source of supply should adequately meet the demand of consumers without any adverse effects on other water users. Water quantity requirements should be based on an assessment of all existing and possible future domestic, institutional, commercial and industrial demands, and should also consider possible water demands for fire fighting purposes.

Water is also a natural resource, which should be managed and used in a responsible manner. Water conservation measures that eliminate water misuse or wastage should be implemented; water metering is a particularly effective method of encouraging the responsible use of water. The use of water saving fixtures, upgrading or replacement of leaking water distribution piping, water use restrictions, and proper water treatment plant operation are other water saving measures which should be considered when determining water requirements.

2.1.2 System Capacity

Various components of waterworks systems should have a design life that is compatible with the function of the component. For example, a water treatment plant should be designed for a minimum period of 10 years with provision for expansion to handle a 20 or 25-year design flow. Intakes and outfall structures, which have high base construction costs, should be designed for the entire design horizon which is at least 20 to 25 years. Storage facilities, on the other hand, should be phased to avoid operational problems (such as increased chlorine demand or oversized pumps) which are associated with excess storage and detention times.

2.1.3 Raw Water Supply and Water Treatment

The raw water supply and water treatment plant should be designed for at least 110% of the projected maximum daily design flow. This compensates for accumulated in-plant losses of up to 10% of the produced treated water.

2.1.4 Water Distribution

Water distribution system should be designed to handle a normal operating pressure between 350 kPa and 550 kPa under a condition of maximum hourly design flow. Distribution system pressures above 550kPa should be reviewed against the Canadian Plumbing code to determine specific building/household requirements to avoid damage to internal building/household piping.

In order to provide adequate service, the minimum distribution pressure during peak demand design flow should be 150 kPa.

In addition to the maximum operating pressures, there are transient pressures due to pump starts and stops, power failures, or rapid valve operation. Pumps should be designed to minimize these surges, and water mains should be designed to withstand these surges, in addition to the maximum operating pressure.

At the discretion of municipalities, distribution mains may be located on lands controlled by the municipality.

2.2 Raw Water Supply

2.2.1 Source Water Protection

Source protection is used to control or minimize the potential for introduction of chemicals or contaminants in source waters, including water used as a source of drinking water. Since almost all water, both surface and groundwater may be a potential source; this really is about the protection of all water resources. Thus, source protection is really the equivalent of, or at least a component of a watershed management plan that usually focus on general (ambient) water quality.The focus in a comprehensive plan will be on contaminants that pose a threat to human health as well as aquatic life.

A waterworks utility needs to have a comprehensive, source-to-tap plan in order to ensure safe drinking water. A source protection plan is only one of the elements of a source-to-tap plan. The utility need not take on the source protection responsibility alone, and in fact, likely can't accomplish it without the involvement and cooperation of others. The source protection component may be achieved in conjunction with other efforts such as watershed management planning and this will serve to perform the first barrier in the multi-barriers in the source-to-tap approach. The watershed management planning process will quite likely involve a wide range of participants including, but not limited to, federal, provincial and municipal governments, watershed groups, stakeholders, and the public. The waterworks utility needs to be a participant in this process as a stakeholder.

In late 2003, the Government of Alberta released *Water for Life: Alberta's Strategy for Sustainability*. Alberta Environment will lead this initiative but the strategy is based on a shift to shared governance through partnerships, including federal, several provincial ministries, municipal and non-government organizations. This strategy will form the framework for province-wide water management planning. The planning areas will be based on watershed scale planning areas. It is this process that waterworks utilities should be preparing to participate.

The goal of a successful plan is to provide guidance in making decisions, by anyone with the authority to do so, so that the outcome is consistent and coordinated. For example, a municipal development authority may use the plan to make decisions on land use. Another example is a provincial government authority making a decision regarding a point-source discharge. The plan communicates the desires, intentions, and possible consequences of decisions.

The waterworks utility's perspective and input into the process needs to be on contaminants that may not be adequately controlled by the treatment system of a waterworks system. Typically, a water treatment system is designed based on the quality of the source water at the time of initial construction or a major upgrade. In order for the treatment system to function well in producing an acceptable product water quality, the source water quality must remain within the anticipated range of quality.

If contaminants are introduced into the system for which the treatment system was never designed or intended to remove, the contaminant has the potential of passing through the process and being present in the finished water. Increased concentrations of contaminants for which the system was designed may result in breakthrough due to exceeding the design criteria. Contaminants that appear and will remain present in the source water after design and construction of the plant effectively reduce the design life of the system. This is a significant impact on the capital investment.

A reduction in contaminants in the source water may have the benefit of improving the finished water quality and potentially extending the design life of the facility thereby providing benefits over and above simply protecting the investment.

A source protection plan must identify the threshold within which the source water is of adequate quality. This is accomplished by reviewing the ability of the selected or existing technology to reduce or remove any given contaminant. Any additional contaminants or concentration of contaminants would therefore threaten the ability of the treatment system to produce adequate quality water.

The plan must examine the source for the existence, or potential for introduction of, contaminants that are above or outside the acceptable range. Once these contaminants are identified, the potential source of the contaminants can be determined and control measures implemented.

Control measures are not limited to eliminating the practice of the activity within the upstream watershed, but may include multiple levels of providing protection that will effectively eliminate the threat and provide satisfactory protection of the finished water quality.

2.2.1.1 Surface Water or Groundwater under Direct Influence of Surface Water Sources

Surface water sources are the most difficult to protect since they may be subject to both point source and non-point-source contaminant introduction. The number and type of contaminants are as diverse as the number of activities that may occur in the watershed upstream of the intake. In addition, the travel time to an intake for any given contaminant is likely to be much shorter than for a groundwater source. This results in the area to be considered within the plan to be extended much further upstream to provide adequate detection and reaction time.

Most conventional surface water treatment systems are designed for removal of suspended sediments and organics such as parasites and microbes. These systems typically are not intended to remove dissolved substances although some incidental removal may be accomplished. The normal method of determining the efficiency of the physical treatment is log reduction of particles. Turbidity is used as an indirect measurement of the number of particles in water. Turbidity reduction to below a specified level is assumed to have achieved the required log reduction.

The initial pre-design water quality characterization must identify the level of log reduction required for the source water. If the value of this parameter in the source water is constant, the system should continue to perform adequately. However, if the parameter is not constant, and over time varies above the initial assessment value, the treatment system will effectively have it's useful life shortened. An unplanned upgrade will be required to maintain the same level treatment as was originally anticipated for the life of the facility.

The presence and baseline concentrations for other chemicals such as pesticides should be evaluated. Although most of these contaminants are likely to be well below levels of concern, monitoring the trend over time will allow review of the adequacy of the protection plan in controlling these chemicals.

Source protection measures may include programs for:

- land use/buffer zones,
- agricultural tillage practices,
- stormwater management,
- material disposal and recycling,
- landfills,
- used oil collection,
- pesticide container collection,
- hazardous waste round up,
- private sewage systems siting, construction and management,
- shoreline and riparian area restoration,
- and many others.

2.2.1.2 Groundwater

Waterworks systems that are served by high quality groundwater sources are not immune to potential contamination and require source protection just like surface sources. The main difference is in identification of the potential points of entry of contaminants into the aquifer. For the most part, these will be point sources such as the wellhead itself, other wells that utilize the same aquifer as a source, or areas of high permeability and recharge. In addition, the delineation of the aquifer may be somewhat more difficult as the extent of the source is not as readily identified.

Each well provides a potential conduit for contaminants into the aquifer. Although control of many of these wells will not rest with the protected system's owner, an education program for private owners will certainly reduce the risk of accidental contamination. Simply knowing the number and types of wells, including their intended use, will provide the ability to identify hazards and prepare contingency plans.

Unused wells also present a hazard to the aquifer. The owners of the wells may be reluctant to permanently abandon (plug and seal) these wells, and wish to retain them as a back-up source. However, the reason for seeking an alternate supply was likely that the well was inadequate in meeting the demands or it suffered partial or complete failure.

Every well, including the protected system's well, whether or not the wells are used regularly or kept for backup, need to be properly sited, constructed, and protected. Wells must be sited on relatively high ground such that water will not pool around the casing. The well construction must ensure that the well is completed in a single aquifer to prevent cross-contamination. The annular space surrounding the casing must be sealed to prevent vertical movement along the casing. The top of the casing must extend above the ground to the highest recorded flood water level.

The presence and baseline concentrations for other dissolved chemicals should be evaluated. Although most of these contaminants are likely to be well below levels of concern, monitoring the trend over time will allow review of the adequacy of the protection plan in controlling these chemicals.

2.2.1.3 Summary

Source protection for waterworks utilities can be accomplished by preparing for, and participating in the *Water for Life; Alberta's Strategy for Sustainability* process. Waterworks owner's can prepare for this process by identifying potential partners, issues and concerns pertinent to their facility, and identifying potential solutions. Each utility will likely share concerns with other neighbouring facilities in the same area and can collaborate to share information and efforts.

Successful development and adoption of a source protection plan, even one that is shared with others, will be of great benefit to the overall source-to-tap protection system.

2.2.2 Water Source/Quality

Raw water from a selected source should be of sufficient quality such that it can be economically treated to produce finished water which complies with the potable water quality (Section 1.1) and the treatment requirements outlined in Section 1.2. Factors that influence the choice of the raw water source should include reliability, treatability, environmental impact, and economics. The choice of filtration process should be based on total coliform count, turbidity and TOC as presented in Table 2.1.

As the level of treatment required would be dependent on the raw water quality, the owners may develop watershed protection programs to reduce any potential risk of source pollution. The owners may maintain a sanitary control area around all sources for the purpose of protecting them from existing and potential sources of contamination. The owners may also develop a watershed control program, identifying land ownership and activities which may adversely affect source water quality and develop watershed control measures, including documentation of ownership and relevant written agreements and monitoring of activities and water quality.

2.2.3 Site Selection Criteria

Several factors that should be considered when selecting a site for new or expanded water supply and treatment works include:

1. isolation from non-compatible land uses;
2. facility location with respect to the raw water source, the area(s) being serviced, and proximity to associated utilities;
3. physical site problems including susceptibility to flooding, subsurface geology, or proximity to natural watershed areas; and
4. adequacy of the site for future expansion.

TABLE 2.1
Generalized Capabilities of Filtration / DAF Systems to Accommodate Raw Water Quality Conditions

Treatment	General Restrictions			
	Total Coliforms (#/100 mL)	Turbidity (NTU)	TOC	Protozoa
Conventional filtration	< 20,000***	No restrictions	No limit	< 3 log
Direct filtration**	< 500	< 7-14	< 3	< 2.5 log
Slow sand filtration	< 800	< 10	< 2	< 3 log
Membrane Filtration	N/A	< 100 NTU	< 2	Based on challenge test
DAF		< 100	No limit	N/A

* Shall insure control of disinfection by-products

** When TOC > 3 mg/L turbidity reduction is impaired

*** When total coliforms > 20,000/100 mL, or TOC > 10 mg/L, additional treatment may be required

Note: Ideally pilot testing should be conducted to demonstrate the efficacy of the treatment alternatives.

2.2.4 Surface Water Supply

2.2.4.1 Intakes

1. Sizing

For the reasons discussed under Section 2.1.2, intakes should be sized for an extended design period (as compared to a phased design), which is usually about 20 to 25 years.

2. Design

Intake design should account for wave action and should provide adequate protection against the effects of ice and boat anchors. Intakes should be identified with buoys or reflectors where in proximity to shipping or recreational activities. The designer should be familiar with the requirements as legislated under the Navigable Waters Act.

The inlet should be located to prevent bottom sediments from being picked up. For small intakes, consideration should be given to providing means for back-flushing the intake, if practical.

The design of river intakes differs from that for lakes and stagnant water bodies in that more secure anchoring is required to resist bottom scouring and stream velocities. River intakes should be equipped with trash racks and should also be located well upstream from potential sources of pollution.

River hydrology must be considered where the riverbed is subject to movement. Design the intake to minimize drawing in Frazil ice.

All intakes in fish bearing waters require Federal Department of Fisheries and Oceans approval.

An acceptable alternative design to direct intake is an infiltration gallery intake. This type of intake is suitable when the riverbed is composed of gravels and rocks or if the floodplain is demonstrated to have a high water table that is connected to the nearby watercourse. Items to be considered are:

a. the sediment load in the river (may necessitate backwashing or aeration provisions);

b. the use of filter cloth; and

c. the depth of perforated infiltration pipes (to be located as deep as possible in the aquifer so as not to be affected by seasonal fluctuations).

2.2.4.2 Screening

1. Sizing

Screen mesh size should be governed by the surface raw water quality and the species of fish present in the raw water supply. Screen size requirements shall be in accordance with requirements of the provincial department of Fish and Wildlife or the federal Department of Fisheries and Oceans.

2. Location and Type of Screen

Screens may be constructed either at the intake structure itself or in-plant just prior to the raw water pumping facilities. For small treatment plants with in-plant screens, two (2) fixed screens in series will suffice, while for larger plants the use of at least two (2) mechanically cleaned screens operating in parallel is recommended. A combination of fixed and mechanically cleaned screens may be used for medium capacity plants. Screens at the intake should comply with the requirements stipulated by the federal and provincial departments of Fish and Wildlife or the federal Department of Fisheries and Oceans.

3. Washing

Fixed screens should have lifting lugs for removal and washing. Screen waste should not be returned to a raw water storage area.

2.2.4.3 Pumping

Pumps should be specified so that the full range of flows anticipated can be provided with pumps operating in the vicinity of their optimum efficiency points, with due regard to the hydraulic design of the discharge piping. This is often accomplished by selecting pumps that have wide band efficiencies and a relatively flat operating curve.

The number of pumps should be consistent with the pattern of flow required and the method of flow control. It is recommended that at least three pumps be provided for operating flexibility; a minimum of two pumps are required, one as standby. Pump capacities should be such that with the largest unit out of service, the remainder will be able to supply the treatment plant capacity as stated in section 2.1.3.

Provision should be made for an individual pressure gauge on each pump and an isolating valve and check valve on the discharge side. Dry well pumps should be provided with suction side valves. The use of slow opening pump discharge should be considered at raw water pumping stations remote from the treatment plant. Piping should be arranged to allow ready disassembly from pump to shut off valves, and include a flexible type coupling to permit proper alignment of the piping and pump. Couplings shall be adequately protected against thrust. Pump elbows should be supported to remove all bending moments, either steady or shock, from pump nozzles.

The station design should allow for future additional pumping units and where possible, the pipe work should be large enough for an increase in pump size to be accommodated. Adequate space should be provided for the installation of these additional units, and to allow safe servicing of all equipment.

Adequate space should be provided for removal of pumps, and in the case of vertical turbine pumps it may be necessary to provide a roof access for removing the units and sectional discharge pipes so that they can be completely removed from the raw water well.

All piping should be arranged so that there is sufficient room to service all valves and other parts, and to permit their removal with minimum disturbance to the system. A bridge crane, monorail, lifting hooks, hoist or other adequate facilities should be provided for servicing or removing equipment.

Pumps should be mounted on bases above the floor level, and all access openings into the well shall have suitable curbs around them to prevent floor drainage entering the well. The station floor should be sloped to floor drains. Floor drainage back to the raw water well is not permitted. Drainage from pumps on to the floor is not acceptable.

The pumps should be capable of supplying the water over the entire range of flows to be treated. This could be achieved through the provision of pumps with variable speed motors or through control valves. Where control valves accomplish this, it is normal practice to use butterfly valves operating in a range to maintain stable control and avoid cavitation. At small treatment plants where substantial seasonal variations in flow exist, it may be necessary to provide duplicate flow control systems - one suitable for very low flows (which normally occur in winter) and one suitable for the plant design flow.

2.2.4.4 Raw Water Storage

Raw water storage improves water quality by providing presedimentation of solids, ensures an adequate supply when a stream or lake source is intermittent, and provides standby against failure of intake facilities. It also enables the operator to avoid the undesirable practice of drawing water during periods of poor raw water quality, allowing a low rate of withdrawal at the source.

1. Facility Planning

   The designer should assess the need, location, and sizing of the raw water storage reservoir before proceeding with final design. Reservoir sizing should be determined by assessing the availability of water and the nature of upstream activities. The designer should also consider any potential adverse effects on the water intake, storage, or treatment facilities; and should include design features to minimize the effects of fluctuating raw water turbidity.

2. Multi-Cell Provision

Raw water reservoirs should be constructed with a minimum of two cells. This will enable the plant operator to withdraw raw water from the second cell when the first cell is being filled or repaired. Each cell should be sized to retain about 75% of the annual raw water needs. In areas of drought, the number and storage capacity of each cell should be increased to overcome long-term droughts.

Control structures should enable the plant operator to isolate each cell, to drain each cell, and to enable the cells to be operated in series or in parallel. A bypass around the reservoirs may also be provided to obtain water during those periods when reservoirs are out of service.

Each cell should be deep enough to restrict light penetration within the depth of the reservoir to discourage the development of ideal habitats for aquatic plants.

Inside slopes of the cells should be armoured, where required, to prevent erosion. The impact of ice formation on winter storage should be accounted for in the design.

3. Reservoir Management

The owners of raw water reservoirs should have a reservoir management program that identifies the current condition of the reservoir, the necessary storage capacity, and the necessary management procedures to respond to changes in reservoir conditions.

The reservoirs should be managed to avoid any difficulties with algae, taste, odour, colour, iron and manganese in drinking water. In-reservoir management techniques should address problems with algae, weeds, low dissolved oxygen, and loss of storage capacity.

Artificial circulation, aeration, phosphorus precipitation, sediment removal, dilution, and flushing are reservoir management techniques that should be adopted to improve the water quality. Use of algaecides is not allowed due to the toxicity of the algaecide and the potential for algae cells to be ruptured resulting in the release of cyanotoxins from certain species of algae.

4. Lining

Raw water reservoirs should be designed to minimize seepage. Reservoirs should be lined in accordance with Section 4.2.1.4 pertaining to wastewater stabilization pond liners. This standard is based upon AENV's publication entitled Design and Construction of Liners for Municipal Wastewater Stabilization Ponds.

2.2.5 Groundwater Supply

2.2.5.1 Siting of Wells

Wells should be located to avoid proximity to sources of pollution and/or flooding. Wells shall be at least 100 m upgradient from pollution sources such as septic tanks, drainage fields, cesspools, or wastewater stabilization ponds; wells should not be located near sanitary landfill sites, underground fuel storage tanks, or cemeteries. Reasonable access shall be provided for repair and maintenance.

2.2.5.2 Well Protection

In order to protect the finished supply structure from external contamination, the following should be provided:

1. Watertight construction to at least 2 m below ground level. This depth may be increased if local conditions present a danger of surface contamination;
2. an annular opening of at least 40 mm outside the protective casing, filled with an approved grouting material; and
3. other precautions in the design to seal off undesirable subsurface formations and surface contamination.

2.2.5.3 Pumphouse Design

The design criteria for well pumping stations generally follow those presented for raw surface water pumping, and standby-pumping facilities should be provided which are capable of maintaining normal servicing standards. In addition, the following special considerations apply:

1. The elevation of the top of the production well casing should be 200 mm above the established ground level or the pumphouse pit floor, and at least 200 mm above the highest recorded flood level;
2. A pump pedestal should be provided around the surface casing to support the full weight of the pump and to prevent any weight from being placed on the production casing or any associated well casing;
3. A water-tight seal should be provided between the pump base plate (or submersible discharge head) and the pump pedestal, and between the well casing and the pump discharge column to prevent the entrance of contaminants;
4. An aperture for air venting with proper screening should be provided to the production well surface casing. Where there are indications of excessive quantities of explosive or toxic gases in the water, both the well casing and pump columns should be vented to the outside of the pumphouse (protection against freezing is required);
5. Return pipes that will permit water to be recirculated down the well should be avoided as they may cause contamination of the well. In cases where recirculation is proposed because of severe water shortages, the proponent should provide design details with the application for a permit;
6. The well should not be located within 1.2 m of an exterior wall of the pumphouse, and should be centred under a hatchway in the roof which is at least 1 (one) metre square to

facilitate access. Also, to accommodate redevelopment of wells, access for service rigs should be provided;

7. Well water quality monitoring should be provided by including a suitable sampling point. Water level monitoring should be provided by including at least one opening in the well head which allows vertical access to the inner casing for equipment installation;
8. Either an electric resistance tape or a water level measuring airline should be installed (clamped to the pump column) complete with a suitably calibrated pressure gauge;
9. The piping layout in the pumphouse should include an in-line free discharge pipe to the outside of the building to permit future testing of the well; and
10. A flow measurement device should be provided.

2.2.5.4 Well Disinfection

Prior to the use of a water well for domestic consumption, the well should be disinfected. Chlorine should be applied to ensure that a concentration of 50 mg/L is present in the well for a period of twelve hours. Dosage should be computed on the basis of water required to provide mixing throughout the entire well volume.

2.3 Water Treatment

2.3.1 Rapid Sand Filtration

The following guidelines have been prepared to document the desirable ranges, and the normal, minimum or maximum acceptable levels for the various design parameters used in the design of rapid sand filtration systems. This document is not a design manual per se as documentation of all parameters relating to water treatment plant design is beyond the scope of these guidelines, but an attempt has been made to include the parameters of greatest importance from the process and reliability standpoints as a guide.

2.3.1.1 General Guidelines

Process Selection

1. Raw Water Characteristics

   The raw water quality is the single most important factor in determining the type and the extent of treatment required for a particular source of water. Thus, a thorough evaluation of the raw water types should precede the selection of a treatment process. The major raw water characteristics are microbiological quality, turbidity, pH, alkalinity, colour, TOC, TSS, iron, manganese, algal counts, temperature and UV absorbance at 254 nm.

   Preferably at least a five-year history that characterizes the main raw water types should be collected. While this is possible for locations where a water treatment plant already exists, it could be impractical for new locations. Therefore, data that characterizes the main water types for at least one year should be collected as a minimum. Facilities that are located upstream and/or downstream from a proposed site may provide valuable information on the raw water characteristics.

2. Enhanced Coagulation

It is recommended that treatment plants using surface waters/GWUDIs should practice enhanced coagulation for disinfection by-products (DBP) precursor removal. TOC content of the source water is generally a good predictor of DBP formation potential, and enhanced coagulation is based on the organic content and driven by the TOC levels. Pre-chlorination should not be practiced at high TOC levels.

Enhanced coagulation is defined by achieving the removal percentages prior to the point of continuous disinfection given the source water alkalinity and TOC concentrations as shown on Table 1.4.

TABLE 1.4: TOC Removal Percentage

Influent TOC mg/L	**Influent Alkalinity, mg/L**		
	0-60	**60-120**	**> 120**
0-2	No action	No action	No action
2-4	40%	30%	20%
4-8	45%	35%	25%
> 8	50%	40%	30%

3. Coagulant Residuals

Aluminium levels in finished water have become a concern for systems using aluminium-based coagulants, particularly for systems producing water with elevated pH. Thus, municipalities should take steps to reduce the amount of aluminium below Health Canada's Operational Guidance Value of 100 ug/L, in the finished water, especially for those plants practicing lime softening.

4. Jar Tests

As bench-scale testing of the treatment often gives meaningful insight to full-scale results, it is recommended that jar tests be done on the raw water. For high TOC waters, jar tests should investigate TOC or UV_{254} removal as well as turbidity removal. Jar test data should statistically establish the treatment requirements of the source water with respect to the process and the water treatment chemicals that are being proposed. The recommended apparatus are a paddle type jar tester, 2.0 litre square beakers, and automatic pipettes. Provisions should be made to ensure that the water temperature during the jar tests is constant and representative of the design conditions. A description of the techniques used, the jar tests results and the conclusions made based on the jar tests should be included in the pre-design report.

5. Pilot Tests

Where practical, pilot studies should be conducted, and specifically when a non-conventional treatment process is proposed or a new water source is being developed. Piloting should be done for sufficient length of time to statistically verify the proposed treatment process. A description of the apparatus, the results and the conclusions made based on the pilot studies should be included in the pre-design report.

Optimization Capability

Unit processes should be designed in order to provide the operator with the flexibility to optimize and integrate each unit. Package plant installations should not be excluded from the requirement to provide the same optimization capabilities as larger plants.

Bypasses to waste should be provided for each process unit and the piping sized for the design capacity of the unit.

Manual or automatic sampling capabilities should be provided at each stage of the process. Multiple sampling locations should be provided for a unit such as a clarifier or flocculator.

Prior to commissioning, a tracer study should be conducted in order to prepare a blueprint of the plant hydraulics that can be used for plant optimization and for troubleshooting future problems. A tracer study should be performed at the initial and at the design flow rate and should quantify actual detention as opposed to theoretical detention times for each operating unit. As an alternative, tracer data supplied by the manufacturer for package plants and proprietary systems would be acceptable. The data from the tracer studies should be included in the as-built plans and/or the operating manual.

For plants with throughput in excess of 10 ML/d, an actual hydraulic grade line at initial and design flow rates should be measured. This data should be submitted with the design hydraulic grade line as part of the as-built plans and/or the operating manual.

Plant Automation

Computer control systems should be provided for all new and upgraded water treatment plants. These will enable the operator to monitor and/or change critical plant operating parameters by means of a computer or a programmable logic controller. Section 2.3.6.3 identifies those parameters for which monitoring may be automated.

2.3.1.2 Chemical Mixing and Coagulation

Chemical mixing is often the first and also an important step in the process train. Mixing is critical for uniform dispersion of the coagulant with the raw water in order to avoid over or under treatment of the water. An understanding of water chemistry and the process of coagulation - flocculation is extremely important in the design of the components of a rapid-mix unit. The water quality, mode of destabilization and the type of coagulant all play a part in the selection and design of the appropriate unit.

Some general comments pertinent for the design of chemical mixing and design criteria for various types of rapid-mix units are as follows:

1. General Comments

    a. The designer should establish the initial and ultimate design flow capacities for each of the different raw water types. The mixers that are selected should be capable of delivering the required energy input for each of these flow rates and raw water types. Unless it can be demonstrated that raw water quality and plant throughput will not significantly change, the rapid mixer should be capable of delivering a range of energy inputs.

b. Simultaneous addition of a primary coagulant and a flocculant aid to a single rapid mixer is not recommended. An optimum time of separation should be derived from jar or pilot tests or should be at a minimum of two minutes. It is not unusual to add flocculant aids at the flocculator.

   Flocculant aids should be mixed into the process stream either mechanically or hydraulically and a maximum energy of mixing should be determined so as not to shear the polymer chain.

c. Dilution of polyaluminum coagulants is not recommended. To avoid precipitation of hydroxide forms, alum should not be diluted to below 0.5% (pH of solution is approximately 3.0) and ferric salts should not be diluted to below 2.5% (pH of solution is approximately 2.0).

d. In-line mechanical mixer should be the option of choice for primary coagulants due to its versatility.

   Mixing systems consisting of a back mix rector or a channel with one or more mechanical flash mixers are not recommended for primary coagulants since they usually result in a lack of instantaneous mixing, short circuiting, and a mixing time that is excessive for metal salts. These systems are acceptable for chemicals other than primary coagulants.

e. Injection of chemicals into a pump suction is not recommended since this results in inadequate control of mixing energy and possible pump damage. Hydraulic rapid mixing of a primary coagulant through venturi meters, parshall flumes, weirs and orifices are also not recommended for primary coagulants since these devices provide inadequate control of mixing energy into the process. They are acceptable for chemicals other than primary coagulants.

f. High energy mixing is not required for chemicals that are not used for coagulation. As a guideline these chemicals can be injected a minimum of 30 pipe diameters or channel widths away from the point of coagulant addition.

The materials used in construction of the chemical mixing system should be corrosion resistant.

Static mixers are not recommended for mixing applications with highly variable flow rates, since at low flow rates, insufficient mixing is often provided.

2. Design Criteria

Rapid Mixing for coagulants
$G > 3{,}000\ s^{-1}$

Rapid Mixing for Polymers
$G = 400$ to $800\ s^{-1}$

3. Scale-up Criteria

Constant value = $N^{5/8} D^{5/9}$
Where:
N = rotational speed, rpm
D = mixer diameter, m

2.3.1.3 Flocculation

Some general comments pertinent for the design of flocculation system and design criteria for various types of flocculators are as follows:

1. General Comments

   a. All systems should employ proper compartmentalization so as to avoid short-circuiting.

   b. Except in very small plants a minimum of two flocculation trains should be provided.

   c. For conventional treatment the flocculation units should be located as close as possible or adjacent to the solids separation units.

   d. The inlet and outlet design should prevent short-circuiting and the destruction of floc.

   e. A drain or pumps should be provided in order to handle sludge removal.

   f. For mechanical flocculators an infinitely variable speed mixer is recommended for the last stage. Two or three speed motors are acceptable for the preceding stages.

   g. Baffles may be used in order to improve the efficiency of mixing. Two or four baffles are recommended and they should penetrate 1/8 to 1/12 of the width across the individual mixer compartment.

   h. Alternative and proprietary designs will be judged on their own merit. Prior to design, pilot testing of the alternative or proprietary systems should be conducted and the results should be included in the pre-design document. As indicated earlier, scale-up based on geometrically similar units with constant $N^{5/8}D^{5/9}$ is recommended.

   i. If there is a significant pH depression, the materials that are used for construction should be corrosion resistant. Alternatively, corrosion control measures (e.g. cathodic protection, coatings) could be used.

   j. Diffused air systems are not recommended due to the high rate of energy consumption and inefficiency. An exception would be if volatile inorganic or organic compounds need to be removed.

   k. Water jet mixing is not recommended due to the high shearing force of the jet that tends to restrict floc size.

   l. Hydraulic flocculators are not recommended for applications with highly variable flow rates; insufficient flocculation energy is often provided.

2. Design Criteria

a. Vertical Turbine Flocculators

$G = 100$ to $10\ s^{-1}$ (tapered)
t = 15 to 40 minutes (for conventional treatment)
G x t = 20,000 to 200,000
Stages = 2 (for direct filtration applications) to 4, typically 3
Maximum tip speed = 2.0 m/s
Blade Area/Tank Area = 0.1% to 0.2%
D/T = 0.2 to 0.4 (D = blade length, T = equivalent tank diameter)
Shaft speed = 8 to 25 RPM

- Recommended for direct filtration (higher energy/shorter detention time) and conventional systems (lower energy/longer detention times).

b. Horizontal Paddle Flocculators

$G = 50$ to $10\ s^{-1}$ (tapered)
t = 30 to 40 minutes
G x t = 20,000 to 110,000
Floc stages = 3 to 2
Maximum tip speed = 1.0 m/s
Blade Area/Tank Area = 5% to 25%
Shaft speed = 1 to 5 RPM
Minimum number of paddles per shaft = 3

- Recommended for conventional treatment systems.

Note: Baffled Walls for Mechanical Mixers

Typical orifices size = 100 mm to 150 mm, rectangular or round, evenly distributed.
Maximum velocity of water through the orifices in the first stage 0.20 m/s
Maximum velocity of water through the orifices in the last stage = 0.35 m/s.

Maximum velocity of water through orifices that connect directly into a sedimentation tank = 0.25 m/s.
Headloss through the ports should be less than 1 cm in order to prevent floc break up.
At the bottom of the baffle, clearance should be provided for the washing and removal of sludge.
The top of the baffle should be slightly submerged in order to promote the passage of scum.
Baffled walls contribute a G of 5 to $25\ s^{-1}$ and this may be incorporated into the overall mixing requirement.

c. Baffled Channel Flocculators or Hydraulic Flocculators

Commonly called "around-the-end" or "over and under"

"G" = 12.7 $(h/t)^{0.5}$ at 4°, where t = residence time in seconds, and h = headloss in metre

G = 50 to 5 s^{-1} (tapered)
t = 20 to 45 minutes
Maximum flow velocity = 1.0 m/s, downstream sections will have much lower velocities.
Typical headloss = 0.2 to 1.0 m
Minimum water depth = 3.3 m
Minimum distance between baffles = 0.75 m
Minimum number of channels = 2
Typical headloss across the tank = 0.3 to 0.2 m

- Recommended for conventional treatment where plant flow variation is small.
- Adjustable baffle walls are recommended for varying raw water conditions.

2.3.1.4 Solids Separation

1. Sedimentation

Some general comments pertinent for the design of sedimentation/clarification system and design criteria for various types are as follows:

a. General Comments

i. The minimum number of sedimentation tanks should be two in separate trains with dedicated filters except in very small plants.

ii. Bottom slope can vary from <1% to 8% depending on the sludge removal method.

iii. Sludge should be automatically removed by mechanical means. Steep sided hoppers shaped bottoms (e.g. >45o) by themselves are inadequate for sludge removal unless the sludge bed is not allowed to compact.

iv. Installation of a grit chamber at the low lift pumps is recommended if raw water contains a large amount of silt and sand.

v. If there is a significant pH depression, the materials that are used for construction should be corrosion resistant. Alternatively corrosion control measures (e.g. cathodic protection, coatings) are recommended.

vi. An overflow to waste should be provided.

vii. A by-pass for the sedimentation unit may be provided to permit direct filtration during maintenance of the tank or when water conditions are suitable for direct filtration. Disinfection should be capable of providing the required inactivation of Giardia and Cryptosporidium during the direct filtration mode.

b. Design Criteria

i. Horizontal-flow type (Rectangular basin without high rate settling modules)

Surface Loading Rate = 0.83 to 2.5 m/h
Water Depth = 3 to 5 m
Detention Time = 1.0 to 3 hours
Length/Width = Minimum 4/1, Recommended 5/1
Maximum Length = 75 m
Maximum Width = 25 m
Freeboard = 0.2 m (typical)
Weir Loading < 11 m^3/m.h.
Reynolds Number <2000
Froude Number >10^{-5}

Due to the potential for short-circuiting and dead spaces, rectangular basins should not be designed with 180° horizontal bends along the flow path.

ii. Horizontal-flow type (Rectangular basin with high rate settling modules)

Surface Loading Rate (Flow/Projected area of basin covered by modules)
= 2.3 to 5.0 m/h (cold water: < 10°C)
= 7.5 to 8.8 m/h (warm water: > 10°C)

Water Depth = 3.2 m (minimum due to mechanical sludge removal) to 5.0 m.
Maximum Area Covered by Tube or Plate Settlers = 75%, from the back wall forward to the inlet.
Weir (Launderer) Loading <15 m^3/m.h.
Approach velocity to modules = 0.2 m/min (typical)

- Higher surface loading rates may be acceptable when a heavy floc is generated. Natural conditions or the use of a flocculant aid can produce rapidly settling floc.
- The surface loading rate is also a function of the type of high rate settling module.
- A continuous sludge removal system is recommended if high rate settlers are employed.
- Due to the potential for short-circuiting and dead spaces, rectangular basins should not be designed with 180° horizontal bends along the flow path.

Note:

1. For perforated Inlet baffles in rectangular basins that are not part of the flocculator

    - Ports should be uniformly distributed across the entire cross section.

- While maintaining the structural integrity of the baffle wall, a maximum number of ports should be provided in order to minimize the velocity of the jets and the dead zones between ports.
- Maximum velocity of the water jet entering the sedimentation tank should be 0.25 m/s.

2. For long rectangular tanks, provision should be made for the design and installation of an intermediate diffuser wall(s). These walls may be installed at or after plant start-up.

3. For Outlets for Rectangular Basins

   - Long finger launderers with or without adjustable weir plates are recommended.
   - Submerged orifice pipes and troughs are acceptable.
   - Significantly higher weir loading rates may be allowed if the weirs are evenly distributed over a substantial portion of the surface (20% to 70%).

iii. Up-flow Type (Radial, without high rate settling modules)

Circular or square in shape
Surface loading rate = 1.3 to 1.9 m/h
Water Depth = 3 to 5 m
Detention time = 1 to 3 hours
Weir Loading < 7 m^3/m.h.

- These are usually proprietary designs.
- Recommended for plants where raw water flow and quality is constant.

iv. Reactor Clarifier

Flocculation time = 20 minutes, and up to 40 minutes in cold water
Surface loading rate < 3 m/h for alum/ferric coagulants
Detention time = 1 to 2 hours
Weir Loading < 15 m^3/m.h.

- These are usually proprietary designs.
- Tube settlers may be added in order to improve performance as plant throughput increases.
- Recommended for water softening or conventional treatment where the raw water quality is constant.

v. Solids Contact Clarifiers (with high rate settling modules)

Flocculation time = 20 minutes typically and up to 40 minutes in cold water
Surface loading rate < 5 m/h for alum and < 9 m/h for lime

Detention time = 1 to 2 hours
Weir Loading = 7.5 to 15 m^3/m.h.
Slurry recirculation rate = 3 to 10 times the raw water inflow rate

- Surface loading rate may be reduced in cold weather conditions
- Lower values for recirculation are used for coagulation.
- Higher values for recirculation are used for lime/soda ash softening.
- These are usually proprietary designs.
- If high rate settling modules are not used, the loading rates would be about a third of what is recommended.
- Recommended for water softening or conventional treatment.

2. Dissolved Air Flotation (DAF)

a. General Comments

i. The minimum number of DAF units should be two.

ii. DAF is a particularly useful process where the floc is light (such as often occurs with low-turbidity raw waters) or where the suspended particles (such as algae) tend to float rather than sink.

iii. Uniform withdrawal of the clarified water, from the bottom area of the units, is recommended to minimize short-circuiting.

iv. DAF should be considered only in conjunction with chemical pre-treatment.

v. Suitability of DAF for a particular raw water source should be substantiated/verified by a pilot study.

b. Design Criteria

i. Bubble diameter = 20 to 100 microns

This is typically achieved by air saturation in a pressurized vessel; larger bubbles will shear rather than float the floc particles.

ii. Surface loading rate = 5 to 15 m/h

Dependent on factors such as water quality and temperature, tank configuration, and size and characteristics of suspended particles.

iii. Saturation pressure = 450 to 725 kPa

Recycle rates = 6 to 10% of the total process flow

Air requirements = 8 to 10 grams/m^3 of raw water

Design of the recycle water injection nozzles shall ensure even distribution across the inlet, and minimize tendencies for the air bubbles coalesce near the point of injection.

iv. Inlet baffle should be placed at an angle greater than 45° to the horizontal (typically 60° to 75°).

Cross flow velocities between the top of the weir and the water surface = 0.7 to 1.0 m/min.

v. Floating sludge removal system - Mechanical removal by reciprocating scraper.

Hydraulic removal will result in dilution of the waste sludge.

2.3.1.5 Filtration

1. Filtration Systems

Where possible, the filtration system should be designed and operated to reduce turbidity levels as low as possible, with a goal of treated water turbidity of less than 0.1 NTU at all times.

There are basically three types of rapid sand filtration systems. They are: declining rate filtration; influent flow splitting; and constant rate filtration.

a. Declining Rate Filtration

In declining rate filtration, filter influent enters through specially designed manifolds to provide virtually equal head to all filters. The filter outlet contains a restriction limiting filtration rate through a clean bed to the maximum allowable rate, and filtration rate declines as the bed plugs. An advantage of this method is that it avoids potential water quality deterioration caused by high shearing forces on a dirty bed which can occur in constant rate filtration as design headloss is approached. Disadvantages include the need for special care in start-up of filters by gradually opening of the effluent valve, and a loss of operating flexibility, which is particularly significant in larger plants.

b. Influent Flow Splitting

Influent flow splitting employs free fall weirs as a means of providing equal flow to multiple filters, thus avoiding the need for, and cost of, effluent control systems. The disadvantages include the potential for flocculated particles to shear passing the weir, thus impairing the filtration process; and that weir settings are normally made to accommodate a single plant flow rate. Changes in plant flow cause different losses in influent channels and the influent flow are no longer equally divided.

c. Constant Rate Filtration

The most commonly used method is known as constant rate filtration. In this method the filter effluent piping contains a flow measuring device, frequently a venturi flow meter, and an automatically controlled modulating butterfly valve. The filtration rate is set to a pre-determined value by positioning the effluent valve accordingly and filtration is continued at that rate by the effluent valve gradually opening to compensate for increased headlosses as the filter bed plugs. The rate is maintained until clearwell level exceeds a level typically 300 mm below the design top water level. At this point the filtration rate decreases proportionally to the 'freeboard' in the clearwell until the filter reaches its lower operating limits at which time it is shut off. This should be set to occur just before the clearwell is full.

2. General Design Consideration

The design of gravity filters should provide:

a. adequate headroom above the filter to permit inspection and operation and maintenance, and provide reasonable access to the filters for observation (e.g., a walkway along the length and width of the filters for package plants),

b. an overflow sized for the filter/plant capacity to prevent flooding, unless provided elsewhere in the raw water supply system and relief from flooding caused by improper operation of the backwash pump,

c. means of cleaning influent pipes or conduits where solids loadings are high,

d. effluent piping designed hydraulically for flows of up to 50% in excess of filtration design capacity to accommodate potential peak demands, provided that the water quality is not compromised,

e. effluent piping arranged to prevent backflow of air into the filter,

f. piping for filter to waste to be designed at full filter flow capacity,

g. design to accommodate measurement of turbidity during production and filter-to-waste modes,

h. operation with a minimum water depth in excess of the design terminal headloss to prevent negative pressure and air binding of the filter as described in subsection 5 – Headloss,

i. an acceptable method of regulating flow as described in subsection 1 - Filtration Systems, and

j. controls as described in Section 2.3.6 - Controls and Instrumentation.

Wash water troughs should be designed so that a clearance is maintained between the bottom of the trough and the level of the expanded media during backwashing.

The bottom of trough to the top level of the static media should not be less than 200 mm, and in some cases should be higher, and the trough capacity should be such that the maximum wash water rate can be accommodated with at least 50 mm freeboard. Trough spacing should be such that each trough serves equal filter areas, and a maximum horizontal travel for suspended particles to reach a trough of 1.0 m is recommended. Troughs should be located so that they do not obscure filters or affect accessibility.

3. Filter Media

The quality of filtered water is a function of both media size and media depth. The selection of filter media determines not only the filter effluent quality but also the filter backwash regime, and thus the backwash requirements become an integral part of the media decision. For instance, the greater the uniformity coefficient of the media, the larger the backwash rate required to fluidize the coarser grains thus provided.

Dual-media filters have some advantages over single-media filters in terms of filter run lengths, headloss, hydraulic loading rate, etc. For dual-media filters, the size of the sand layer shall be selected to be compatible with the anthracite that has been selected. The bottom sand layer should have approximately the same or somewhat higher flow rate for fluidization than the anthracite to ensure that the entire bed fluidizes at the selected backwash rate. Further, the effective sizes of the sand and anthracite should be selected to achieve the goal of coarse-to-fine filtration without causing excessive media mixing.

The particle size and the depth of the media selected for a given filter application depends on the kind of suspended solids to be removed by the filter. This is best established by pilot studies. Typical design data is shown in Table 2.3.

Sieve analyses should be performed and the values plotted to ascertain sand size distribution.

Deep bed filtration, i.e. > 2.0 m, may be used in conjunction with the policies and procedures outlined in AENV's policy - Unproven or Innovative/Alternative Technologies.

4. Filtration Rates

Filter loading rate is an important parameter in the design of a water treatment plant; and in general the filters may be designed to be operated with the loading rate in the range 2 m/h to 9 m/h.

Low filtration rates do not ensure good quality of water; what is more critical is the chemical pre-treatment and the filter design. With adequate chemical pre-treatment and filter design, filtration rate of up to 15 m/h may be applied without deterioration of the filtrate quality. However, high-rate filtration (i.e. 9 m/h < rate < 15 m/h) should be substantiated by a filter column study using the proposed source water under the full range of anticipated water conditions.

Filtration rates higher than 15 m/h may be used only in conjunction with the policies and procedures outlined in AENV's policy - Unproven or Innovative/Alternative Technologies.

TABLE 2.3: Typical Design Data For Dual-Media Filters

	RANGE	**TYPICAL**
Anthracite		
depth, mm	300 to 600	450
effective size, mm	0.8 to 2.0	1.2
uniformity coefficient	1.3 to 1.8	1.5
Sand		
depth, mm	150 to 300	300
effective size, mm	0.4 to 0.8	0.5
uniformity coefficient	1.2 to 1.2	1.4
Ratio of coal size to sand size (D_{90} coal/D_{10} sand)	-	3

5. Headloss

The total head available or head loss on a filter is the difference in elevation between the water levels on the inlet and outlet side of the filters. This is a critical parameter for influent flow splitting and constant rate filters. For a specific media and flow rate, the total headloss and the time to reach a fixed headloss depend on the volume of floc retained by the filter. Thus headloss is closely associated with the filter run lengths; operation of a filter with long filter runs and high headlosses will result in break-through of the flocs.

Typical headloss for gravity filtration may be in the range from 0.3 m (clean bed) to 2.5 m (final). The filters may be operated with run lengths between 12 and 72 hours, with 24-hour runs being typical.

If the headloss at any level in the filter bed exceeds the static head, a vacuum is created, resulting in air binding in the zone of negative pressure. In order to eliminate the problem of negative pressure, the filter should be designed to discharge the effluent water at a level above the sand media surface.

6. Filter Backwash

Three basic methods are available for filter backwash: Up flow water wash without auxiliary scour, up flow water wash with surface wash, and up flow water wash with air scour. The suitability of a washing method is related to influent water quality, filtering media and bed configuration, and underdrain design. Consequently, not all washing methods are applicable in all cases, and different methods may or may not yield similar results in a particular case. Declining rate backwash systems are not acceptable.

Required backwashing rates are variable and depend on water temperature, filter type and washing method. Water velocity required to achieve the same bed expansion increases as the temperature increases, thus backwash systems should be designed for the warmest wash water temperature.

a. Up flow Water Wash Without Auxiliary Scour

For the washing to be effective, backwash rate should be sufficient to fluidize the bed with 30 to 40% expansion. The relatively weak cleaning action of water wash without auxiliary scour generally renders it unsuitable for filters removing large quantities of suspended solids.

When water wash is used alone, a rise rate of 32 m/h to 54 m/h should be applied. Backwash duration may vary between 3 to 15 minutes, depending on how dirty the filter is.

b. Up flow Water Wash With Surface Wash

Surface wash is generally applied to supplement up flow backwash where mud ball formation is likely to be a problem. Either a fixed-nozzle or rotary wash system may be used; rotary systems are preferred because they provide better cleaning action, lower water requirements, and less obstruction for filter access.

For optimum results, a combined surface and water wash should be designed to be operated in three phases. First, surface wash is activated and operated alone for 1 to 3 minutes. Water wash is then applied simultaneously at a low rate to achieve a bed expansion of less than 10 percent for 5 to 10 minutes. The final phase involves application of wash water only at a higher rate to achieve 30 to 40 percent bed expansion for 1 to 5 minutes.

Rotary surface wash system should be designed to add 1.5 m/h to 2.0 m/h to the wash water flow; and, fixed nozzle system to deliver 2 m/h to 9 m/h.

Because surface wash systems constitute a possible connection between filtered and unfiltered water, backflow prevention devices shall be provided in supply lines.

c. Up flow Water Wash With Air Scour

There are three approaches to use auxiliary air scour in backwashing filters.

Air scour alone for 3 to 5 minutes followed by low-rate water wash for 5 to 15 minutes could be applied for single media-filters with 0.2 to 1.2 mm media. Air scour should be designed to inject air at 18 to 27 $m^3/m^2.h$; and water wash at 12 m/h to 18 m/h.

The second approach, which is suitable for dual-media filters, is to apply air scour alone for 3 to 5 minutes followed by water wash for 5 to 10 minutes. Bed stratification would occur during backwash cycle. The air scour should be designed to inject air at 54 to 90 $m^3/m^2.h$, and water wash at 12 m/h to 18 m/h.

The third approach is to have simultaneous air scour and water wash followed by water wash alone. During the initial stage, air scour rate of 18 to 90 $m^3/m^2.h$ should be used with a water flow of about 18 m/h for approximately 5 to 10 minutes. This is followed by water wash only for about 5 to 10 minutes at a rate of one to two times that of the previous cycle used with air scour.

2.3.1.6 Flow Measurements

Flow meters should be provided on:

1. the main raw water supply line to measure the total flow entering the plant;

2. individual filters. This may be necessary for constant rate filtration to ensure that each filter receives the same flow; and for declining rate filtration to initiate backwashing of filters;

3. the combined filter effluent line and/or on the distribution main head to measure water usage by the community; and

4. the backwash line to measure and check the volumes of water used for backwashing the filters.

2.3.2 Slow Sand Filtration

The slow sand filter is a sand filter operated at very low filtration rates, frequently without the use of coagulation in pre-treatment. The sand used is smaller than that for a rapid filter, and this, plus the low filtration rate, results in the solids being removed almost entirely in a thin layer on the top of the sand bed. This layer, composed of dirt and micro and macroorganisms from the water (i.e. the schmutzdecke), becomes the dominant filter medium as the filter cycle progresses.

When the head loss becomes excessive, the filter shall be cleaned by draining it below the sand surface and physically removing the schmutzdecke. Typical cycle lengths may vary from 3 to 6 months depending on the source water quality and the filtration rate.

2.3.2.1 Requisite Conditions

Source water quality and community size are the key factors that should be considered in the selection of slow sand filtration to treat surface water.

1. Water Quality

   Raw water quality determines the length of time between scraping options, and slow sand filtration shall be limited to raw waters that will permit filter runs of at least three months before terminal headloss is reached.

   The mix of water quality characteristics causing headloss is unique to each situation and cycle lengths cannot be predicted without pilot plant testing. Unless specifically exempt by AENV, suitability of slow sand filtration for treatment and run-length shall be ascertained by pilot testing to cover all four seasons of the year.

   Some waters with high algae content form a mat on the filter surface causing a rapid increase in headloss. Further, the decay of algae on the schmutzdecke may cause taste and odour problems in the effluent water. Thus source waters subject to algae blooms may not be acceptable for slow sand filtration unless the bloom season is short enough that only minimal interruption of normal operation is likely.

   Though low turbidity of source water is not always indicative of long filter run-lengths, water with high turbidities can blind off the filter with a layer of sediments on top of the filter bed. Thus source waters with high turbidities may not be acceptable for slow sand filtration, unless the turbidities are attenuated by the use of sedimentation basins or roughing filters.

   Slow sand filtration is not effective in removing colour, and source waters with high colour are not acceptable for slow sand filtration.

   Slow sand filtration is ineffective in removing dissolved organic carbon, and because of potential disinfectant by-products, source waters with high dissolved organic carbon (DOC) may not be acceptable for slow sand filtration if chlorination is practised for disinfection. However, in some cases, ozone ahead of slow sand filtration has proven to increase run length and increase biodegradation of organic compounds, i.e. DOC converted to BDOC. This is considered an alternative technology and will be approved only in conjunction with the policy, "Unproven or Innovative/Alternative Technologies" in the Municipal Policies and Procedures Manual.

2. Community Size

Slow sand filtration is considered appropriate for use by "small" communities. At some point in population size, slow sand filtration becomes more expensive than rapid sand filtration. Also, at some point as population increases, communities will have the requisite resources to effectively and reliably operate a rapid sand filtration system. The point of crossover on both curves depends upon the community context, and should be considered in ascertaining the suitability of slow sand filtration for that community.

2.3.2.2 Filter Media

Ideally the effective size (D_{10}) of the sand should be just small enough to ensure a good quality effluent and to prevent penetration of clogging matter to such a depth that it cannot be removed by surface scraping.

The recommended sand size is D_{10} = 0.2 to 0.3 mm with uniformity coefficient (UC) = 1.5 to 2.0. Media having a sand size with $D_{10} \geq 0.3$ mm and UC >2 may be acceptable if a pilot study ascertains that acceptable removals are obtained.

The sand being considered should be washed to remove dust/fine particles to avoid long start-up times. Sieve analysis should be performed and the values plotted to ascertain sand size distribution.

2.3.2.3 Filter Depth

The depth of sand bed is determined by the number of years of operation desired before re-sanding is needed. The years of operation are calculated as follows:

$$Y = \frac{Di - Df}{R.f}$$

in which
Y = period of operation, yr
Di = initial sand bed depth, mm
Df = final sand bed depth, mm
R = sand depth removal per scraping, mm
f = frequency of scraping, number/yr

Generally sand bed depth should be between 1 to 1.3 m in depth at the start of operation, a deeper bed may be used if desired. The minimum bed depth should be greater than 0.5 m, assuming the filter is biologically mature. A staff gauge to measure sand bed elevations should be placed on the wall/bank of the filter to denote minimum and maximum sand levels.

2.3.2.4 Filtration Rates

Hydraulic loading rate for the maximum daily design flow may vary between 0.1 m/h and 0.4 m/h. The system should be designed so that when one of the filters is removed from operation for scraping, the combined loading rate of the remaining filters may not exceed the design filter hydraulic loading rate. While the hydraulic loading rate may vary during the annual cycle and generally increase as the population grows, the flow should be steady over the daily cycle.

2.3.2.5 Headloss

Headloss within a slow sand filter is caused by flow through the schmutzdecke and the sand bed; but mostly through the schmutzdecke.

The clean bed headloss is usually about 0.01 m, depending on the hydraulic loading rate, the temperature and the sand media characteristics. Acceptable headloss limit is economic rather than technical because there is no evidence that higher headloss would cause breakthroughs.

Typical headloss for slow sand filtration may be in the range from 0.1 m (clean bed) to 2.0 m (final). The rate of headloss increase is crucial to the determination of whether slow sand should be selected as the filtration technology because the length of filter runs depend on the rate of headloss increase. This can be analyzed only by pilot plant testing. In general, the rate of headloss increases with time, and the marginal benefits of continuing the filter run decrease rapidly after a certain period.

2.3.2.6 Hydraulics

1. Distribution

   Delivery of the entire flow to a filter to one point in the filter bed would result in bed erosion. The consequence of such a condition is short-circuiting of flow through the sand bed. To control bed erosion, the inlet should be designed to distribute and dissipate the kinetic energy. One approach is to distribute the influent flow around the filter bed; the lateral pipe should be large enough that the exit velocity would be sufficiently low. Distribution pipe should be placed approximately 0.3 m from the top of sand bed. For open basins, the impacts of ice cover must be considered.

2. Collection

   Filters should always operate with a uniform hydraulic loading rate over the sand bed and this is dependent on underdrain collection system.

   Collection of the filtered water is through slotted or perforated pipes surrounded by gravel support. The underdrain system should be designed using the manifold hydraulic principle; that is, the headloss within the main pipe should be small compared with the headloss through the orifices in the main pipe. If this principle were maintained, then the hydraulic loading rate across the filter bed would be uniform.

   Spacing of the underdrain varies from 1.0 m to 2.0 m; closer spacing is preferred, as it would provide an added certainty of uniform loading of the filter.

3. Drainage

   To scrape the sand bed, the headwater shall be drained to a level just below the sand bed surface. Provision should be made in the design to drain the top portion of the headwater directly either through the influent pipes or through a separate pipe.

4. Backfilling after Scraping

After scraping the de-watered filter shall be backfilled with treated water. Provision should be made in the design to backfill the filter through the underdrain system; backfilling from the top can result in entrapment of air bubble which may cause air binding and disruption of the flow.

The treated water may come from an adjacent operating filter or from the clearwater tank. Water from clearwater tank, if used, should be free of chlorine. The headwater should be backfilled to the level of the influent distribution orifices, which is usually placed about 0.3 m above the sand bed to provide the water cushion. Valves and pipings should be provided to accomplish the aforementioned tasks.

5. Flow Control

Flow to the plant should be controlled on the influent side downstream from the influent flow meter. The flow into the plant should be steady over a twenty-four hour period. The treated water storage tank should have an overflow weir with drainage pipeline to waste or recycle, if continuous operation of the filters is desired.

6. Tailwater Control

Tailwater control is necessary so that the water level in the filter can be raised to about 0.3 m above the sand bed immediately after scraping in order to dissipate the kinetic energy of the influent flow.

The design of the system should have a movable weir plate that can be raised or lowered during operation. The initial position of the weir crest, at the start of a filter run, should be about 0.3 m above the top of the sand bed, but the plate should be adjustable enough that it can be lowered to the elevation of the surface of the sand bed once the head loss across the sand bed is greater than 0.3 m.

7. Headloss Measurements

Piezometers should always be installed in filters to measure head-loss. One piezometer should be connected to the headwater above the sand bed, and a second one to the tailwater basin. The piezometers should be 25 to 50 mm in diameter mounted side-by-side, with float balls and scales, to permit easy measurements of water levels.

8. Avoiding Negative Pressures

Negative pressures within the filter bed cause the formation of gas bubbles, which may cause "air binding" and thereby disrupt the flow patterns of water movement through the sand bed.

In order to avoid negative pressures, the system should be designed so that the weir crest for the tailwater elevation control may not be lower than the level of the sand bed during operation.

2.3.2.7 Gravel Support

The function of the gravel support is to support the sand bed and to permit uniform drainage of the overlying sand. In order to have uniform drainage and minimal headloss, the gravel support shall be graded with finer material at the top and coarser material at the bottom.

1. Size

   The gravel support should be designed so that the top layer of the gravel support should not permit migration of sand from the sand bed, nor should the gravel of any layer find its way to a lower level. The bottom layer should not permit entry of gravel to the underdrain orifices. The following rules should be followed in the design of the gravel support layers:

   a. d_{90} (given layer) / d_{10} (given layer) ≤ 1.4
   b. d_{10} (lower layer) / d_{10} (upper layer) ≤ 4
   c. d_{10} (top layer) / d_{15} (sand) ≥ 4
   d. d_{10} (top layer) / d_{85} (sand) ≤ 4
   e. d_{10} (bottom layer) $\geq 2.d$ (drain orifice diameter).

2. Depth

   The thickness of each layer should be greater than three times the diameter of the largest stone. At the same time, as a practical matter, the minimum thickness of gravel layers should be 50 to 70 mm for finer material and 80 to 120 mm for coarser gravel.

2.3.2.8 Flow Measurements

Flow meters should be provided on:

1. the influent side for the whole plant; and
2. volumetric flow meter on the exit side for the whole plant.

   The flow meters for the individual filters are used to ensure that each filter receives the same flow and to measure the volumes of water filtered between scrapings. The volumetric flow meter would indicate the amount of water used by the community.

Aesthetic Objectives

Potable water quality should meet the Aesthetic Objectives (AO) in the Guidelines for Canadian Drinking Water Quality, published by Health Canada, as amended or replaced from time to time.

The water distribution system should be cleaned and flushed periodically to keep the turbidity less than 5 NTU.

2.3.4 Lead and Copper

Waterworks system should produce non-corrosive water to minimize lead and copper corrosiveness. It is recommended that the owners conduct corrosion control studies unless they can show that corrosion control is already optimized. Corrosion control studies should compare the effectiveness of pH and alkalinity adjustment, calcium adjustment, and addition of a phosphate or silica-based corrosion inhibitor.

The owners should work with AENV to establish a protocol for optimizing corrosion control.

2.3.5 Iron and Manganese Control

Iron and manganese exist in water in both the insoluble and soluble oxidation states. The insoluble iron and manganese are readily removed by filtration. However, the removal of soluble iron and manganese poses a more serious problem, particularly when these metals are organically bound.

Presence of soluble iron and manganese in water may lead to complaints from the consumers as a result of staining of laundry or bathroom fixtures. Water containing iron and manganese also promotes the growth of iron bacteria in mains, with accompanying increases in friction loss.

2.3.5.1 Removal of Iron and Manganese

1. Oxidation/Filtration

   Removal of soluble iron and manganese by contact adsorption using pre-coated filter media is the option of choice for treating waters with moderate amounts of iron and manganese in groundwater (<5 mg/L of iron and <1 mg/L of manganese). The filter media that could be used are:

   a. Manganese greensand (natural greensand coated with manganese dioxide);

   b. Pyrolusite (pure manganese dioxide).

   Key to the success of contact oxidation process is the re-generation of manganese dioxide coating on the media, either on a continuous basis or on an intermittent basis.

   Dual media filtration is recommended if iron removal is the main objective. Continuous regeneration operation is also recommended where iron removal is the main objective with or without the presence of manganese. This method involves the feeding of a pre-determined amount of an oxidant (potassium permanganate or chlorine), directly to the raw water prior to the filters. If chlorine is used for waters containing ammonia, sufficient chlorine should be fed to go beyond breakpoint to produce free chlorine for regeneration of the media. The oxidant demand should also be determined taking into consideration the presence of DOC and H_2S.

TABLE 2.4: Design Criteria for Iron and Manganese Removal

Design Parameter	Main Component To Be Removed	
	Iron	Manganese
Regeneration of media	Continuous	Intermittent
Bed type	Dual media	Single media
Depth of bed	Anthracite - 375 to 450 mm MnO_2 media - 450 to 200 mm	MnO_2 media > 750 mm
pH	2.2 to 8.5	7.0 to 8.5
Filter loading rate *	4 to 13 m/h	4 to 13 m/h
Headloss	1.5 m (maximum)	1.5 m (maximum)
Backwash	Sufficient for 40% bed expansion	Sufficient for 40% bed expansion

*a. The higher the concentration of iron and manganese, the lower the loading rates for equivalent run lengths. For optimum design parameters a pilot plant shall be operated.
*b. For intermittent regeneration process, higher loading rate of up to 2.0 m/h may be allowed depending on raw water conditions.

Intermittent regeneration is recommended for waters where only manganese or manganese with small amounts of iron is to be removed. This method involves feeding of a pre-determined amount of an oxidant after a specified quantity of water has been treated. Frequently, intermittent regeneration is done during the backwash cycle. Typical design criteria for iron and manganese removal using oxidation/filtration process are shown in Table 2.4.

2. Aeration/Chemically Assisted Filtration

Aeration followed by chemically assisted filtration is effective in treating waters with fairly high content of iron (>5 mg/L) and manganese (>1 mg/L). Without coagulation and flocculation, the oxidized iron may take 12 to 24 hours or more for effective settling, whereas in properly coagulated water, settling will take place in approximately two hours. If manganese is present, pre-treatment with one or more oxidants (chlorine, chlorine dioxide, potassium permanganate or ozone) is required.

Design criteria is similar to the requirement outlined in Section 2.3.1 - Rapid Sand Filtration.

2.3.5.2 Control of Iron and Manganese

Iron and manganese concentrations in drinking water may be controlled by blending water from different sources so that the combined iron and manganese concentrations are below the aesthetic objectives stipulated in the GCDWQ.

If iron and/or manganese control with sequestering agents is proposed, the total concentrations may remain greater than the aesthetic objectives. Sequestering is generally ineffective with hard waters; pilot testing may be required to verify the performance of the proposal.

2.3.6 Controls and Instrumentation

2.3.6.1 General

Controls and instrumentation should be appropriate for the plant size, complexity, criticality, and number of staff and their skills for each plant. To achieve this, the designer should develop a control philosophy that will enable the plant staff to effectively monitor and control the plant and major equipment, the treatment process, water production and; plant wastes. The following design guidelines represent a minimum standard that shall be achieved. Specific enhancements may be required depending on the water treatment process employed.

2.3.6.2 Measurement List

For plants of 1 ML/d capacity and greater, the following instruments should be provided as a minimum for the relevant processes listed. For smaller plants, pH measurement and fluoride residual may be by bench testing but all other instruments are appropriate for the relevant processes listed.

1. Raw Water Instrumentation:
    - Low-level switches to shut down the raw water pumps. These should be hard-wired to the starters
    - Running and trip indication for raw water pumps
    - Raw water turbidity, pH, pressure, flow rate, and flow volume
2. Rapid Mixer:
    - Running and trip indication
3. Flocculators:
    - Running and trip indication
    - Speed if variable speed type
4. Solids Contact Clarifiers:
    - Recirculator speed indication
    - Running and trip indication
    - Level indication
    - Blow down valve status

    Turbidity and pH following clarification
5. Proprietary Clarifiers

    Instrumentation for proprietary types of clarifiers including ballasted flocculation and DAF should be as recommended by the manufacturer. However, effluent turbidity and pH are recommended in all cases.

6. Softening:

- If lime softening is used, pH following recarbonation

Recarbonation CO_2 flow present

7. Filter (granular) Instrumentation:

- Turbidity on each individual filter effluent and filter to waste. This can be a single instrument for each filter if piping arrangement permits.
- For constant rate filters: differential head loss across the filter media
- Filter flow rate
- Provide open and close limit switches or position on all filter valves and status on backwash equipment

Filter run time

Filter effluent particle counters are optional but are encouraged.

8. Filter (membrane) Instrumentation:

Instrumentation is normally packaged in with the system to meet the manufacturer's needs. However, the following are recommended regardless:

Turbidity and particle counters on each individual filter train effluent.

Filter train flow rate

Pressure differential

- Filter train status (on-line, backwash, clean, off-line etc.)

9. Backwash Instrumentation:

- Running and trip indication for backwash pump(s)
- Running and trip indication for air blowers (if air scour is used)
- Backwash flow rate and flow total
- Surge protection for air blowers

10. Clearwell & Distribution Pump Instrumentation:

- Level indication for clearwell and other tanks
- Low-level switches to shut down the distribution pumps. These should be hard wired to the motor starters.
- Turbidity, chlorine residual, fluoride residual (if fluoridation is practised), pH, pressure, flow rate, and flow total on plant discharge.
- For variable speed pumps, indicate the pump speed

11. Chemical Systems:

- Running and trip indication for chemical loading, batching and pumping equipment
- Low and high level indication in storage bins, silos or tanks

- Level indication for tanks
- Weigh scales for hydrofluosilicic acid day tanks or storage if no day tank is used
- Weigh scales for gaseous feed chemicals such as chlorine or sulphur dioxide
- Speed indication on variable speed pumps
- Rotameters for carrier water feed systems
- Chemical feed flow rate is mandatory unless day tank is provided
- Chlorine alarm
- UV – provided by the vendor but some additional instruments are usually required such as flow meters and valve status for each reactor.

12. Miscellaneous Instrumentation:
    - Run time meters on all pumps and major electrically driven equipment
    - Speed, run time, oil pressure and temperature gauges, fault signal switches and manual start and shut down on engines
    - Where the plant is automated or operated remotely from either within the plant or outside, provide open and close limit switches or position on all major valves, status on all major equipment and security instruments including door switches, remote resets, building temperature switches and smoke alarms.
    - Any additional instrumentation recommended by equipment manufacturers.

2.3.6.3 Degree of Automation in Plant Control

The control system may be manual or automatic or a combination thereof. Regardless, the system should be designed to promote safety of staff and the public, energy efficiency, conserve water, and reduce waste while meeting the treated water quality standards and demands under all anticipated conditions. Discuss the operating philosophy with plant staff and owners to determine the appropriate degree of automation.

In the case of a manual system, all equipment is started and stopped by the operator, all backwash sequences and other process operations are controlled by the operator, and chemical and pump rates are manually adjusted. This requires that the plant be attended continuously while in operation, perhaps with more than one operator.

In the case of an automatic system, all equipment is started and stopped by the control system, with chemical feed rates and pump rates adjusted automatically to maintain the system levels, discharge pressures, etc. This may allow unattended plant operation or operation with few staff, but requires a more complex and expensive control system, with associated maintenance. Provide the ability to manually operate all equipment.

There are a wide range of combinations of manual and automatic control systems. For conventional process systems with rapidly varying raw water conditions, fully automated plants should not normally be considered.

2.3.6.4 Alarms and Status Indication

All alarms shall be latched until the Operator has acknowledged them. If the alarm is indicated by a lamp, it shall flash until acknowledged then remain steady until the alarm clears. If it is indicated on a computer screen, an appropriate color code or symbol shall be used to indicate for each alarm whether it has been acknowledged. Automated systems should log the time at which the alarm occurred, the time it was acknowledged and the time it cleared. Logs may be printed on paper or recorded electronically.

Valve and equipment status should use a consistent method of symbols and colours, whether the status is indicated through lamps or on a color computer screen. Be certain that the color-coding scheme is consistent with any existing equipment displays elsewhere in the plant.

As a minimum, provide the following alarms:

- high turbidity on the raw water, clarifier effluent (if applicable), filter effluent, and plant discharge
- high and low pressure on the raw water line
- high flow rate on the raw water line
- high and low water level in clarifiers or flocculators
- high torque on process rotating elements (e.g., basin mixers, flocculators, solids contact clarifier recirculator and rakes)
- high water level in process basins and open surface channels
- high and low level in chemical storage tanks
- high and low chemical feed rates
- high flow rate on each individual filter (also low flow rate on declining rate filters)
- high and low water levels in each clearwell, pumpwell, and reservoir
- high and low pH on the raw and treated water (if on-line measurements are provided)
- high and low chlorine residual on the plant discharge (where on-line measurements are provided)
- high head loss on the filters (if influent flow splitter or constant rate type)
- trip or failure to run on each pump and process motors
- high and low pressure on the plant discharge line
- high flow rate on the plant discharge line
- chlorine gas detection in the chlorine storage, metering and injector rooms
- chlorine scale low weight (where scales are equipped with transmitters)
- valve operation failure (where valves are provided with limit switches).

More alarms may be required where additional treatment processes are provided. Alarms should be provided for all control system interlocks that can shut down equipment or systems. In plants that are left unattended for periods of time, provide an automatic telephone dialler, cellular communication or pager system for annunciation of alarms.

2.3.6.5 Control Equipment (Automatic Systems)

1. Programmable Logic Controllers and Distributed Controls Systems.

   Automatic systems should use either Programmable Logic Controllers (PLCs) or a Distributed Control System (DCS). The operator interface may be in the form of traditional control panels (i.e.: lights, gauges and switches), electronic control panels (with text and/or graphics) and computers.

2. Operator Interface

   The operator interface may consist of a local hard wired control panel or mimic, character based input/output panel, personal computer or workstation depending on system size, process complexity, control system functions and operator interface manufacturer. Where personal computers or workstations are used, select the hardware based on reliability, software compatibility, vendor support and suitability for continuous operation in the plant environment. Provide the hardware and software necessary to facilitate back up of both the system and the collected data. The operator interface software may provide the operator with interactive control and monitoring of the plant, handle and annunciate alarms, log and trend events and process variables and generate the required reports. Process control and logic should be performed by the PLC or DCS and not the operator interface computer or workstation. Where outside access is provided for remote control of the operator interface or other control components (e.g., PLC) provide adequate security.

3. Communication Networks

   Plant communication networks shall be considered. Installing communication networks may assist operations and maintenance personnel with the following functions:

   - device diagnostic data is available
   - remote calibration is possible
   - device alarm status is available
   - distributed control by locating PLCs or RTUs in close proximity to devices being monitored
   - plant data can be made available for use by management
   - loop error is reduced.

Plant communication networks can be broken into four types, Sensor Networks, Device Networks, Control Networks and Enterprise networks. Sensor and Device networks are primarily focused on communication with primary elements such as push buttons, temperature transmitters and valve actuators. Control networks provide backbone communication between computers, PLCs and DCSs. Enterprise networks are, for example, local area networks (LAN) and are used to provide information that help to manage the plant. An example of an Enterprise network would be providing data in a spreadsheet format that can be used to assist management in filling out reports for regulatory monitoring.

2.3.6.6 Network Security

Control system security should be considered for mitigation of internal and external threats. Plant internal security at a minimum should include password protection to the control system, allowing only authorized users into development software, plant control software and enterprise software. Control systems should be connected only if suitable hardware and software security devices are in place to prevent unauthorized access to the system. Control systems that are connected to an intranet or wireless system should include provisions to restrict access to the control system by unauthorized personnel.

After the initial installation, the security system should be monitored and upgraded regularly. Improvements should include the upgrading of firewalls and routers, changing passwords and teaching plant personnel on the importance of network security.

2.3.6.7 Historical Data

The control system's historical data should be stored electronically and backed up on removable media such as tape or compact disk. The historical data, and the necessary hardware and software to access the data, should be kept for a period of time as defined in section 1.11.1 - Record Keeping.

Historical data collection should include:

- all parameters required to be monitored in accordance with an Approval or a Code of Practice or defined elsewhere in these documents;
- alarms; and
- operator changes, such as set points or control modes.

Data collection frequency will depend on the parameter being monitored and how quickly the parameter changes. At a minimum, data should be collected at a steady time interval that will allow for recreation of parameter without significant error. Various historian software packages have built-in computer algorithms that will minimize data collection while still maintaining the parameter's trend. The collection frequency should be adjusted to allow for proper tracking of the parameter, e.g. seasonal adjustment of raw water turbidity during spring run off.

2.3.7 Colour Codes for Water Treatment Plant Piping

TABLE 2.5: Recommended Colour Coding for Water Treatment Plant Piping

Piping to be Identified	Basic Colour	Bands	
		No.	Colour
Raw or unfinished water	Dark Green	-	-
Clarified Water	Dark Green	1	Black
Filtered Water	Dark Green	2	Black
Filtered and Chlorinated (Potable) Water	Blue	1	-
Backwash Water	Light Green	-	-
Chemical Feed Lines	Pink	-	-
Coagulant	Pink	1	Black
pH Control	Pink	2	Black
Taste and Odour	Pink	3	Black
Fluoride	Pink	1	Green
Chlorine and Water	Pink	1	Yellow
Chlorine Gas	Yellow	-	-
Plumbing (Waste)	Brown	-	-
Electrical	Purple	-	-
Compressed Air	White	-	-
Heating	Silver	-	-
Fire Protection	Red	-	-
Natural Gas	Orange	-	-

Notes:

1. Entire length of pipe to be painted in basic colour.
2. Bands, if required, are to be placed as follows:
   (a) at 9 m intervals, and/or
   (b) where the pipe enters and leaves a room.
3. Individual bands are to be 25 mm wide, and a 25 mm space is to be left between bands where multiple bands are required.

2.3.8 Laboratory Requirements

All water treatment plants should have laboratory facilities for analytical testing as required in the terms and conditions of the operating approval. The capability of the laboratory should commensurate with the size and the complexity of water treatment operation. Before undertaking the detailed design of the laboratory facility, contact should be made with AENV to confirm the testing requirements.

Minimum laboratory requirements for surface water treatment plants are as follows:

1. **Lab equipment**

    pH - pH meter
    Chlorine - chlorine residual titrator or colorimetric test kit
    Turbidity - turbidimeter
    Colour - colour meter (spectrophotometer or colour comparator)

    Tap aspirator/filter apparatus for colour samples
    Glassware (beakers, measuring cylinders, sample bottles, flasks, pipettes, burettes)
    Balance (top-loading) - if any chemical has to be weighed out
    Clean lab bench area, with sink, adequate lighting, and storage.

2. **Lab chemicals/standards**

    pH butter solutions for calibration
    Hydrochloric acid for cleaning pH probes and titrators
    Distilled water or reagent grade water for rinsing
    Turbidity standards
    Colour standards
    Reagents for any colorimetric tests
    Lab grade dishwashing detergent.

2.3.9 Operational Monitoring

Operational monitoring and associated reporting requirements would be established on a site-specific basis. The nature of the water supply source, the type of treatment system employed and the size/capabilities of the owner should be considered when establishing operational monitoring requirements. Operational monitoring requirements are established both for specific process control purposes, and to ensure that a facility receives good operational attention on a regular basis.

Table 2.6 outlines the operational monitoring requirements that would apply to waterworks systems.

TABLE 2.6: Operational Monitoring Requirements

Parameter	Point of Measurement	Requirement/Objective	Minimum Monitoring
Raw water turbidity	Before addition of any chemical or treatment process	None	See Section 1.10.3.3
Treated water turbidity	Immediately after filtration before entering the Clearwater tank	See Section 1.3.1	See Section 1.10.3.3
Raw water flows	Entering the treatment plant	Not to exceed treatment plant's design capacity	Once per day for total daily flow
Treated water flows	Entering the Clearwater tank or the distribution system	None	Once per day for total daily flow
Raw water Giardia levels	Entering the treatment plant	None	See Section 1.2.1.1
Raw water pH	Before addition of any chemical	None	Once per day using grab sampling
Raw water iron and manganese	Before addition of any chemical	None	Once per week using grab sampling
Chemicals used	Feed point	The chemical dosage should not exceed the recommended maximum concentration authorized by AENV	Volume/Weight/Concentration of chemicals used daily or weekly
Treated water pH	Entering the distribution system	6.5-8.5 (exceptions are acceptable if disinfection is not compromised)	Once per day using grab sampling
Treated water aluminum (if aluminum based coagulant is used)	Entering the distribution system	Not to exceed 100 ug/L for conventional plants, and not to exceed 200 ug/L for other types of treatment	Once per week using grab sampling
Turbidity within distribution system	Random location throughout the distribution system	Not to exceed 5 NTU	Once per week using grab sampling
Treated water iron	Immediately after filtration before entering the Clearwater tank	Not to exceed 0.3 mg/l	Five times per week, twenty-four hours apart using grab sampling
Treated water manganese	Immediately after filtration before entering the Clearwater tank	Not to exceed 0.05 mg/l	Five times per week, twenty-four hours apart using grab sampling

Notes:
Monitoring of these parameters is not required during the calendar days the treatment plant is not operated or during statutory or civic holidays.
Raw and treated water turbidity monitoring are also required from a compliance point of view.
Raw water Giardia levels will be based on the running annual average of monthly samples over a two-year period (See Table 1.1)
Raw water flow rates should be reported in m^3/d.
Specified monitoring for iron and manganese is required only for plants treating for these parameters.

2.3.10 Water Treatment Chemicals

2.3.10.1 Labels and Material Safety Data Sheets

Federal and provincial legislation requires hazardous products at worksites to be labeled and information be made available to workers through Material Safety Data Sheets. It also requires employers to train workers and workers to be knowledgeable with the Workplace Hazardous Materials Information System (WHMIS).

Most chemicals used for water treatment are "controlled products". "Controlled products", hazardous products at the worksite that meet certain criteria, will either be labelled with a supplier label or worksite label, in accordance with the requirements specified in the latest edition of WHMIS.

For appropriate use of the chemical, water treatment operators should be aware of the chemical purity (concentration), shelf life, expiry data, and maximum dosage and use restrictions. This information can usually be found on the supplier labels but may be added to the worksite label for ease in use.

More specific information on the hazardous ingredients, hazards, health and safety risks, safe handling instructions, emergency and first aid measures, are contained on a Material Safety Data Sheet (MSDS). MSDS obtained from the supplier and not more than three years old shall be available at the worksite for all "controlled products" unless it is laboratory product where the label may contain all the information required on a MSDS.

Storage buildings and outside storage tanks should be labelled or placarded with the name of the product and/or hazards of the products.

2.3.10.2 Storage and Handling

1. General Provisions

   Storage should be provided for at least thirty days of consumption at the maximum anticipated chemical usage rate, allowing for variations in chemical dosage and flow in that period. Alternatively, the systems should have a program in place to ensure unlimited supply of those chemicals. Storage capacity for essential chemicals should be at least sixty days, with ninety days preferred. Where deliveries of chemicals may be interrupted by adverse weather conditions in isolated locations, provision should be made for increased storage capacity taking into consideration that some chemicals degrade with time, e.g. NaOCL. Where deliveries at short notice can be assured and the material is not essential to the production of safe water, then requirements may be reduced. If practical, there should also be sufficient storage space to accommodate 'full load' deliveries.

   Chemical storage areas should be segregated from the main areas of the treatment plant, with separate storage areas provided for each chemical. Where chemicals in storage may react dangerously with other materials in storage, e.g. chlorine, fluoride and ammonia or strong acids and alkalis, segregated storage should be provided. The storage and feed equipment areas should be arranged for convenience of operation and observation, and located to provide easy access for chemical deliveries.

It is strongly recommended that all chemical storage be at or above the surrounding grade. Where subsurface locations for chemical storage tanks are proposed, these locations should be free from sources of possible contamination, having drainage for ground waters, chemical spills, and overflows. Where above grade storage is provided, due consideration should be given to the method of unloading chemicals. In general, storage areas should be arranged to prevent any chemical spills. Floor surfaces should be smooth and impervious, slip-proof, and sloped so as to drain rapidly; walls and floors should be protected with a chemical-resistant finish. Storage areas should have eye-wash and/or deluge shower facilities, adequate facilities for cleaning up chemical spills, space for cleaning and storage of the recommended protective equipment, and adequate warning signs to identify hazards. It is recommended that all doors in chemical buildings open outward, and that corridors or space between storage areas be a minimum of 1.5 m wide to permit the safe movement of materials.

Chemical ventilation systems should be arranged so that air is exhausted outside the building and also so that slight negative pressures are maintained where dry chemicals are in use, as a dust control measure. Where large amounts of dust are anticipated, appropriate local exhaust systems and filters or scrubbers should be provided in the ventilation system. Ventilation systems should be designed specifically for corrosive service, and special measures taken in dust systems to prevent build-up of static or other explosive conditions.

2. Liquid Chemicals

All bulk storage tanks should have an adequately sized fill line, sloped to drain into the tank. The fill line should be adequately identified at the end that is remote from the tank, and provision should be made to drain this fill line if required.

Each tank should have an adequate vent line, with a down-turned end. Where venting outside the room is required, the vent should be provided with an insect screen.

All tanks should have an overflow which is adequate for the rate of fill proposed for the tank, sloped down from the tank with a down turned end with free discharge located where it can be readily noticed. Overflow shall not be directly discharged into storm drainage system or to a watercourse. If discharged into the sanitary sewer system, the overflow pipes shall not connect directly to the sewer, and where they pass into a receiving sump or conduit they shall terminate at least two pipe diameters above the maximum level in the sump. Each tank should also be provided with means to indicate the level of contents in the tank, and where an external level gauge is provided a shut-off valve at the tank connection is recommended.

All storage tanks should also be surrounded by a structure to contain the volume of the largest tank in the event of a rupture or spill.

In small treatment plants, day-tanks should be provided for the liquid chemicals. Flow meters should be provided for all liquid chemical systems without day tanks.

3. Dry Chemicals

Where dry chemicals are to be used, provision should be made to minimize handling and dust problems. From a materials handling perspective, granular materials should be preferred to powders.

Particular care should be taken to protect workers and mechanical and electrical equipment from fine dust. Where exhaust fans, filters, and conveying systems are used, grounding should be provided to prevent the build-up of static electricity. Dust control equipment should also be protected against moisture accumulation.

Bulk storage silos should be provided with adequately sized fill openings, and fill lines where necessary should be smooth internally with long radius elbows. Silos should have suitable level indicating devices and shall be equipped with a pressure relief valve when pneumatic fill systems are provided.

If powdered activated carbon (PAC) is used, spark-free lighting and electrical systems should be provided. Provision shall also be made to scrub or filter the carrier air, when dry PAC is off-loaded into silos.

4. Hypochlorite Solution

Municipalities should purchase hypochlorite solution with low levels of chlorate. Steps should be taken to avoid formation of chlorate ion in the solution. Hypochlorite solutions should:

- contain less than 1500 mg chlorate/L;
- have a pH greater than 12;
- be used within a relatively short time frame after delivery (within 3 months);
- be stored in a cool dry location where the temperature does not exceed 30°C, away from sunlight; and
- contain less than 0.08 mg/L of transition metals.

Manufacturers are able to produce bleach that has a lower initial concentration of chlorate; municipalities should specify hypochlorite solutions with a chlorate concentration as low as possible to ensure that they will meet the Canadian drinking water quality guideline.

2.4 Water Transmission and Distribution Mains

Where local municipal standards and guidelines for transmission and distribution mains exist, then those standards and guidelines may be followed.

2.4.1 Pipe Sizing

Water mains designed to carry fire flows should have a minimum inside diameter of 150 mm. For smaller distribution systems without fire flow provision, hydraulic calculations should verify that the proposed pipe sizes are sufficient to sustain the minimum operating pressure as defined in Section 2.1.4.

2.4.2 Valves and Hydrants

2.4.2.1 Valve Placement

Water distribution systems should have shut-off valves located to allow any pipeline to be isolated for repairs.

Air release valves should be placed at all significant high points in the transmission system, and should also be considered at high points in the water distribution system. In addition, drain valves should be placed at low points of large mains to permit drainage during repairs to the distribution system.

2.4.2.2 Fire Flows and Hydrants

The provision of fire protection is solely the decision of the Local Authority.

Where hydrants are provided, the leads should be valved for easy maintenance. Where groundwater levels are above the hydrant drain port, the drains should be plugged and the barrels pumped dry for winter conditions.

For details regarding fire protection requirements in municipal waterworks system design, the designer should refer to the most current Fire Underwriters Survey publication entitled Water Supply for Public Fire Protection - A Guide to Recommended Practice.

2.4.3 Pumping

In general, the requirements for treated water pumping station are similar to those outlined in Section 2.2.4.3 for raw water pumping station.

The distribution system by pumping should be designed with at least two pumps. With one pump out of service, the remaining pumps should be able to deliver the maximum hourly design flow at not less than 150 kPa.

In order to supply water economically during low demand periods, at least one pump should be provided with a variable speed motor or an appropriately sized, small pump may be installed.

Standby power or an auxiliary gas or diesel powered pump should be provided to supply water during power outages or other emergencies. Fuel should be stored above ground and outside the water treatment plant building in compliance with the security assessment plan.

2.5 Potable Water Storage

2.5.1 Sizing

The total water storage requirements for a given water supply system where the treatment plant is only capable of satisfying the maximum daily design flow may be calculated using the following empirical formula:

S = A + B + (the greater of C or D)

where S = Total storage requirement, m^3
A = Fire storage, m^3
B = Equalization storage (approximately 25% of projected maximum daily design flow), m^3
C = Emergency storage (minimum of 15% of projected average daily design flow), m^3
D = Disinfection contact time (T_{10}) storage to meet the CT requirements, m^3, as detailed in 1.10.3.7.

The level of fire protection is the responsibility of the municipality. The level of storage may be further reduced if the water treatment plant is capable of supplying more than the maximum daily design flow or if there is sufficient flow data to support a lower peaking factor than would be normally used for the given population range.

The designer should recognize that the given formula for calculating treated water storage requirements shall be supplemented with the storage required for the operation of the water treatment facility, i.e. filter backwash and domestic use.

2.5.2 Phasing

Treated water storage requirements should be calculated using a first phase projected demand of no more than 10 years (refer to Section 2.1.2). Present worth cost analysis may show that longer design periods are more economical, however the failure to properly phase the storage requirements can result in operational problems due to oversized pumping facilities and/or problems with maintaining the required chlorine residual if the available storage is excessive.

2.5.3 Alternative Types

There are many alternative methods for the provision of treated water storage at either the water treatment plant or in the water distribution system. The choice of underground, ground level, or elevated storage will depend on factors such as the service area size, topography, and economics. Each alternative should be investigated to choose the best overall method of storage based on specific project conditions.

2.5.4 Site Selection

An economical site selection will depend on the type of reservoir, but in general the major factors to consider are soils conditions, compatibility with future expansion requirements, and site access.

In cases where it may be necessary to provide two-way flow direction during periods of high demand, it is recommended that the storage facility be located on the opposite site of the high demand centre from that at which the supply facilities discharge to the distribution system.

2.5.5 Design Considerations

2.5.5.1 Multi-Cell Provision

Underground or ground level storage reservoirs should be constructed with two cells for ease of maintenance. This can often be accomplished as a result of phasing requirements. Flexibility should be built in to operate the cells in series or parallel; or pump from either cell. When planning the type of reservoir, the designer shall ensure that treated water is not stored or conveyed in a compartment adjacent to untreated water if the two compartments are separated by a single wall.

2.5.5.2 Access

Treated water storage reservoirs should have convenient access to the interior for cleaning and maintenance. Ensure access is secure.

2.5.5.3 Vents

Reservoirs should be vented by specially designed vent structures. On ground level structures, vents should terminate in an inverted 'U' construction, the opening of which is at least 200 mm above the roof or sod, having a cover using suitable non-corrodible screen cloth. Design or enclose vents to prevent the possibility of contaminants being introduced either accidentally or through a malicious act.

2.5.5.4 Drains and Overflows

Where feasible, storage reservoirs should be provided with a drain. There shall be no direct connection between any storage reservoir drain or overflow and a storm or sanitary sewer. Overflows should be located at sufficient elevation to prevent the entrance of surface water. A backflow preventer should be installed on all overflows.

Freezing of overflow lines may result in rising water levels, frozen vents, and collapse of the reservoir by vacuum action. To prevent this occurrence, overflows should be designed with a large inlet weir to allow a substantial flow that is less likely to freeze. It is also desirable to have the overflow discharge to a warm environment.

2.5.5.5 Circulation

It is most desirable to have perfect plug flow through the clear water tank at the water treatment plant. Such a design would ensure that all water entering the tank had the same detention time to ensure a high a T_{10} value (> 0.6). Baffling design and inlet and outlet design can lead to a near plug flow operation. Refer to Appendix D for illustration of typical baffling conditions.

Balancing reservoirs within the distribution system should be designed to achieve complete mixed flow to avoid poor mixing and circulation, poor turnover rate and excessive detention time.

2.6 Watering Point

Watering point can be either a truck fill station or a barrel fill station.

The truck fill supply should have a minimum pumping capacity of 1000 L/min to minimize the truck fill time; a separate pump should be provided for barrel fill supply. All water supplied from the facility should be metered.

An exterior overhead truck fill arm should be installed. The design shall be such that there will be no cross contamination during or after filling the truck or barrel. If the truck fill is operated in freezing conditions, drain the area to minimize ice build-up due to spillage.

An exterior coin meter activated pump control should be installed so that no person needs to enter the building. The panel should include start and stop buttons, a pump selector switch, and an adjustable maximum run timer or volume control.

2.7 Water Systems Security and Protection

Water systems have been identified as part of the Critical Infrastructure of the province and they must be protected from the threat of terrorism and other forms of criminal behaviour. The following provisions will ensure that water systems operators have accurately assessed the threats to their water systems and introduced appropriate mitigations to ensure continuity of operation and the safety of the public.

2.7.1 Vulnerability Assessment

The waterworks system serving a population greater than 25,000 people should undertake a Security Vulnerability Self Assessment every two years, and systems serving a population less than 25,000 people should undertake the assessment every five years. The system owner / operator should provide certification of performance of the self-assessment submitted to AENV for review and follow up.

(See Appendix G for details)

2.7.2 Emergency Reaction Plans

The system owner / operator should develop an Emergency Reaction Plan (ERP), it is recommended that the ERP should be developed to the standard set out by The Canadian Standards Institute in CSA z 731. This ERP should be based on the results of the vulnerability assessment and should include response to natural disasters, terrorist acts, and other acts of criminality and system failures.

Emergency Reaction Plans should be reviewed and updated annually to ensure contact-information is accurate and the specified resources remain available.

Emergency Reaction Plans should be tested regularly to ensure appropriateness and deliverability. Every test should be documented including any lessons learned and the plan should be updated if required.

2.7.3. Physical Security

The system owner / operator should implement physical protection systems and measures to protect their investment in capital assets and ensure that the safety and security of the public water systems is not compromised by criminal acts.

Physical Protection Systems should incorporate security policies, designs, barriers, devices and systems that offer high degrees of Deterrence, Detection, Delay and Response. These are the basic security principles and can be defined as follows

Deterrence – Make the target unattractive to an adversary by increasing the perception of security. This can be achieved by use of signage, access control and overt uniformed security.

Detection - ensures that if an adversary does decide to attack an asset, that they are detected as early as possible. Detection is achieved by alarm systems, CCTV, Security guards, public and staff.

Delay – Security should incorporate a layered approach with a number of barriers to impede the attacker's progress. Delay is essential to allow for an appropriate response. Layered security is achieved by fencing, locked doors, access control and good design.

Response – Having detected an adversary there should be an appropriate response. In some cases this is achieved by remote assessment using cameras, in other cases this is a response by security guards or plant staff.

2.7.3.1 Personnel

The system owner / operator should consider appointing a Security Manager or other senior manager whose prime responsibility is addressing the security needs of the water utility. This person would be responsible for completion of the vulnerability assessment, implementation of appropriate security measures and delivery of security awareness training. The security manager should be supported by senior management and be adequately resourced.

The system owner / operator should consider initial and periodic background checks on employees and on-site contractors. Appropriate policies should be developed and implemented governing the performance of such screening and the criteria for adverse employment decisions.

2.7.3.2 Training

The system owner / operator should develop and deliver security awareness training to all staff on a regular basis. This training should include awareness on the Emergency Reaction Plans and specific roles and responsibilities contained within the plan.

The aim of security training is to establish a culture of security within the water utility. All staff are responsible for the security of the system and security considerations should be built into all operations. All staff should be aware of the role they play in securing the assets, from ensuring doors are locked, to challenging strangers and reporting or investigating suspicious incidents.

2.7.3.3 Access Control

The system owner / operator should establish some form of access control to its facilities; especially those concerned with water treatment or treated water storage. Access control can take many forms from simple key control to installation of electronic card access systems. The objective of access control is to prevent unauthorised personnel from gaining access to critical assets or components of the water system.

The following are examples of access controls:

- use of non-reproducible high security keys that should be signed in and out on a daily basis. Issuance of master keys should be limited and all keys should be accounted for by an audit on a regular basis.
- use of ID cards to identify all authorized staff and contractors
- use of Security Guards to validate the credentials of all staff, contractor and other visitors
- use of electronic card access or biometric systems to gain access to facilities.

2.7.3.4 Perimeter Security

Water treatment facilities should be fenced to exclude the public from the facility and to protect the water system from unlawful intrusion. Establishing a fence also creates a defensible space for the installation of intrusion detection systems and will deter an opportunistic intruder and provide a platform for the detection of a determined intruder.

Fences should be at least seven (7) feet high topped with three strands of barbed wire. Fences should have a minimum of three (3) metres of clear zone on either side to remove anything, which may hide an adversary or assist them in compromising the fence. Fences should be maintained to ensure they have adequate tension and that the fabric is not suffering from corrosion.

Fences should also be considered for raw water sources, wellheads, and treated water storage and distribution components such as pumping stations.

2.7.3.5 Detection Systems

The system owners / operators of water treatment plants and facilities should ensure that intrusion detection systems (alarm systems) are installed to detect an unlawful intruder. Such systems should be monitored by a ULC Certified monitoring company or by the municipalities own security staff.

Consideration should be given to the installation of exterior intrusion detection systems, such as:

- fence line intrusion detection
- exterior passive infra-red sensors
- microwave sensors
- buried magnetic sensitive cable
- digital closed circuit television systems.

Consideration should be given to the installation of intrusion detection systems onto treated water storage access hatches.

2.7.4 SCADA Security

The municipalities / system owners / operators should take steps to ensure the availability, confidentiality and integrity of computer systems used to control water treatment, distribution or quality monitoring. Where possible, SCADA systems should not be on the same network as other business or municipal computing systems.

2.7.4.1 Critical Cyber Assets

The system owners / operators should quantify their critical cyber assets, that is the components of their SCADA network, without which the ability to deliver on their mission is compromised.

Critical cyber assets should be protected, physically and logically, from the threat of unauthorised access.

2.7.4.2 Identification of Connections

All network connections on a SCADA system that utilise a routable protocol should be identified. These connections should be configured to prevent unauthorized access.

2.7.4.3 Authentication and Access

All SCADA systems should be configured to require strong authentication of users. This can be achieved through the use of passwords that are changed periodically and require alphanumeric content. Other methods of authentication can be considered including use of tokens or biometric authentication.

Legacy passwords and old user accounts should be deleted.

Users should assign the minimum level of access required for them to do their job. Administrator or super user type access should be highly restricted and should not be the default user level.

2.7.4.4 Anti-Virus

The municipalities / system owners / operators should consider installation of anti-virus applications on SCADA systems which have network connectivity. Anti-virus systems should be kept updated with the most current signature files.

2.7.4.5 Operating Systems

The system owners / operators should ensure that SCADA operating systems are kept up to date with current patches. Operators should consider development of a SCADA test environment to test newly released patches prior to deployment in the SCADA production environment.

2.7.4.6 Network Logging

The municipalities / system owners / operators should install logging services onto their SCADA systems to track activity such as, users log-on, rejected log-on attempts, user log-off, unauthenticated user sessions, system faults and intrusion attempts.

Network intrusion detection systems should also be considered, but only if the resources are available to monitor and react to network incidents.

2.7.5 Excavations and Open Trenches

In order to ensure public safety, Local Authorities responsible for the construction shall secure excavations and open trenches during non-working periods by installing fences/barricades and/or warning lights/signs.

2.7.6 Reservoirs

To guard against trespassing and vandalism, treated water storage reservoirs should be fenced, and entrance gate, access manholes, and valves or vent houses should be locked.

The reservoirs should also have water-tight roofs and hatches, and any openings should have suitable covers to prevent the entrance of birds, animals, insects or dust and be designed or enclosed to prevent the possibility of contaminants being introduced either accidentally or through a malicious act. Consideration should be given to the installation of secondary hatches and intrusion detection systems.

Where a raw water reservoir is located near developed areas or agricultural land, fencing is required to discourage attempts by children to gain entry and to preclude the entrance of domestic animals. Fencing should be located outside the toe of the berm; chain link fence is recommended. Fishing will not be permitted in raw water reservoirs.

2.8 Facility Risk Assessment

The municipalities / system owner /operator may undertake a self-assessment of the facility every two years to establish the risk factor. The self-assessment report should be submitted to AENV for review and follow-up. Facility assessment is necessary to assure the general public and AENV that municipalities / system owners/operators are committed to take the required action to reduce any potential risk in providing safe drinking water to the consumer.

System owners / operators may use the criteria tabled below for facility risk assessment.

Table 2.7
Waterworks System Assessment Criteria

Source		Treatment Assessment		Monitoring and Operational Assessment
Surface water	**Groundwater**	**GWUDI**	**Groundwater**	**Surface water and Groundwater**
Quantity - Can the source provide a secure, long-term supply of raw water? If yes, is the raw water storage system adequate? - Have there been trends in supply variability over the past 10 years?	Quantity - Has hydrogeological assessment been done to determine the long-term feasibility of the source? If yes, is the supply adequate? If no, has there been a history of well yield problems over the past 10 years?	Design Criteria - Does the existing treatment process meet current design standards related to the following: - chemical feed and mix - flocculation - clarification - sedimentation - filtration (media, membrane, loading rates, filter-to-waste, backwash, etc)	Design Criteria - Does the system include components for the treatment of health-related or aesthetic parameters? If yes, does the system meet the treatment standards? - Is the existing system adequate to provide for 4-log reduction of viruses? - Is sufficient disinfection residual provided in the distribution system on a consistent basis?	Monitoring - Does the system provide on-line, continuous monitoring for disinfectant residual (all systems) and turbidity (for surface water supply systems only)? If yes, is there an alarm method to provide the operator with immediate notification if pre-set limits are exceeded?
Quality - Has an assessment of the raw water supply been done to determine cysts/oocysts removal requirements? - Are there persistent raw water quality problems related to organics, colour, parasites, or blue/green algae. - Does the raw water quality fluctuate frequently?	Quality - Has an assessment of the water quality been done? - Is the supply suspected to be under the influence of surface water (GWUDI)? - Are there health-related parameters for which treatment is required? - Are there aesthetic parameters for which treatment is required?	Capacity - Does the system produce adequate quantities of treated water to meet maximum daily flow? Will the capacity be fully utilized in the next 10 years?	Capacity - Does the system produce adequate quantities of treated water to meet the standards for maximum daily flow? Will the capacity be fully utilized in the next 10 years?	System Operation - Does the system owner employ the facility certified operator directly, or is an operator provided under contract? - Is there a succession plan in place to ensure continued operation by a certified operator? - Has there been a recent history of the owner having difficulties maintaining a certified operator? - Has the system owner prepared an emergency response plan to deal with emerging problems related to the water source, treatment, or distribution system?

Table 2.8
Waterworks System Risk Factor Source Assessment Summary

Source	1 (minimum risk)	2	Ranking Criteria 3	4	5 (high risk)	Assigned Rank
Quantity	Assessment done. Adequate for present and future needs	No assessment done. Appears to be adequate for present needs	Assessment done. Adequate for present needs but not for future needs	No Assessment done. Appears inadequate for present needs	Assessment done. Inadequate for present and future needs	
Quality	Water quality assessment done, no problems confirmed	Water quality assessment done, no apparent problems	No water quality assessment done, no apparent problems	No water quality assessment done, problems evident	Water quality assessment done, confirmed problems	
Source Protection	Assessment done, no mitigative measures needed	Assessment done, but action taken to mitigate issues	No assessment done, no apparent concerns	No assessment done, problems evident but no action taken	Assessment done, problem confirmed, no action taken to alleviate problems identified	
Overall Rank						

Table 2.9
Treatment Assessment Summary

Treatment	1 (minimum risk)	2	Ranking Criteria 3	4	5 (high risk)	Assigned Rank
Design criteria	The plant meets the design standards	Rating not used	Rating not used	Rating not used	One or more critical component not meeting design standards	
Capacity	Evaluation done. The plant operates within design capacity at present and for future needs	Evaluation done. The plant operates within design capacity at present, but would not meet future needs.	No evaluation done, no immediate concerns	Evaluation done, intermittent, shortages identified, nearly at 90% capacity	Evaluation done, plant has exceeded capacity, persistent shortages at present	
Performance - Filtration (Surface & GWUDI)	Individual filter turbidity < 0.1 NTU at all times	Individual filter turbidity < 0.3 NTU at all times	Combined filter turbidity < 0.3 NTU at all times	Combined filter turbidity < 0.3 NTU at least 95% of the time	Combined filter turbidity > 0.3 NTU more than 5% of the time	
Performance - Fe & Mn removal (Groundwater)	Meets 0.05 mg/L manganese & 0.3 mg/L iron	Rating not used	Meets only one parameter (iron or manganese)	Rating not used	Does not meet 0.05 mg./L manganese & 0.3 mg/L iron	
Performance - Disinfection + filtration: meets 3-log cysts, 3-log oocysts and 4-log virus reduction (Surface & GWUDI)	Meets the prescribed CT / IT requirement, and meets THM standard	Rating not used	Meets CT requirement but not the THM standards	Meets the THM standards but not the CT requirement	Does not meet CT or THM standards	
Performance – Disinfection: meets 4-log reduction virus reduction (Groundwater)	Meets the prescribed CT / IT requirement, and meets THM standard	Rating not used	Meets CT requirement but not the THM standards	Meets the THM standards but not the CT requirement	Does not meet CT or THM standards	
Overall Rank						

Table 2.10
Monitoring and Operation Assessment Summary

Operations Management	1 (minimum risk)	2	Ranking Criteria 3	4	5 (high risk)	Assigned Rank
Monitoring turbidity	On-line, individual filter and combined filter	Rating not used	Online combined filter only	Rating not used	Grab, combined filter only	
Monitoring Disinfection (at point where CT is estimated)	Continuous on-line	Rating not used	Grab daily	Rating not used	Grab less than daily	
Operations (Facility)	Certified operator at appropriate level, cover off, succession plan available	Rating not used	Certified operated at appropriate level, no cover off, no succession plan available	Rating not used	No certified operator at appropriate level, no cover off, no succession plan available	
Operations (SOP/ERP)	SOP/ERP in place	Rating not used	Either SOP or ERP in place but not both	Rating not used	SOP/ERP not available	
Overall Rank						

2.9 Conservation and Reclamation

The system owner / operator shall ensure that the requirements outlined in the AENV documents *Reclamation Assessment Criteria for Pipelines*, 2001 Draft (Alberta Pipeline Environmental Steering Committee, 2001), as amended, as well as the *Reclamation Criteria for Wellsites, Batteries and Associated Facilities – 1995 Update*, as amended, are met.

Achieving pipeline reclamation success requires an understanding, and the sound management of topographic, soil and vegetation characteristics. Equivalent land capability, based on a comparison of pre-activity (pre-disturbance or control data) versus reclaimed land capabilities on that right-of-way, is considered the standard at which a pipeline operator has met the requisite land surface conditions. . Other non-land conditions may also apply. Equivalent land capability is defined as "the ability of the land to support various land uses after conservation and reclamation that are similar to those existing prior to an activity being conducted on the land, but that the individual land uses will not necessarily be identical". System owner / operator shall contact the Regional Director and the landowner and obtain information regarding appropriate land capabilities, subsequent land uses and revegetation strategies.

The three primary land components requiring reclamation management (topography, soil and vegetation) generally co-vary such that changes in vegetation growth patterns, health and population structure are often attributable to changes in soil and topographic characteristics.

TOPOGRAPHY

Surface contour and drainage re-establishment along the pipeline alignment shall be reclaimed to be comparable to pre-disturbance land conditions.

SOIL

Soil Salvage and Handling

Topsoil, upper subsoil and spoil shall be salvaged and stockpiled separately, unless otherwise authorized. Two-lift soil handling can provide a means to salvage upper subsoil and spoil together, provided no "problem" soils (e.g., saline, sodic, rocky) are encountered. When conditions permit the use of overstripping soil handling procedures, all salvaged topsoil admixed with upper subsoil shall be stored separately from spoil.

Topsoil and upper subsoil salvage operations shall be suspended when wet or frozen field conditions, wind velocities, or any other field condition or pipeline construction method may result in soil quality degradation or loss. Operations shall only recommence when those conditions no longer exist. Refer to *DRAFT AENV Manual on Soil Conservation and Pipeline Construction* (1995), as amended, for further guidance.

Soil Storage

Topsoil, upper subsoil and spoil are required to be stockpiled separately. When two-lift soil handling procedures are used, upper subsoil and spoil may be stockpiled together. When conditions permit the use of overstripping soil handling procedures, all salvaged topsoil admixed with upper subsoil shall be stored separately from spoil.

All stockpiles shall be stabilized to prevent wind and water erosion, and constructed with stable foundations.

Soil Replacement

The system owner / operator shall make every effort to progressively reclaim the pipeline alignment when construction is completed, unless the timing of clean up and reclamation have been scheduled to accommodate changes in soil conditions (e.g., frozen).

Salvaged spoil are required to be replaced in the trench and contoured appropriately such that all upper subsoil is spread evenly over the replaced spoil, and that all topsoil shall be spread evenly and contoured over the replaced upper subsoil or spoil.

Topsoil and upper subsoil replacement operations shall be suspended when wet or frozen field conditions, wind velocities, or any other field condition or pipeline construction method will result in the degradation or loss of soil quality. Operations shall only recommence when those conditions no longer exist.

Upper subsoil and spoil compaction shall be alleviated through mechanically breaking, fracturing or shattering prior to topsoil replacement. The approval holder shall also alleviate replaced topsoil compaction in a manner controlled to avoid topsoil mixing with upper subsoil or spoil. Refer to *DRAFT AENV Manual on Soil Conservation and Pipeline Construction* (1995), as amended, for further guidance.

The size and density of coarse fragments (i.e. gravel, rock or stones) within the reclaimed soil profile and on the reclaimed pipeline alignment surface shall be comparable with the adjacent undisturbed land. In forested areas, rollback shall be placed for access management and erosion control.

All works, buildings, structures, facilities, equipment, apparatus or machinery used during construction or reclamation that will not be used for the operation of the pipeline shall be dismantled and removed. All constructed access roads for the pipeline alignment that are not remaining as surface improvements or as a designated property access shall be reclaimed.

VEGETATION

The pipeline alignment shall be stabilized; surface contoured and revegetated using a suitable seed mixture. An effective weed control program shall be in place until new vegetation on the pipeline alignment is re-established and is self-sustaining.

Where the pipeline alignment crosses native prairie areas, native seed mixtures and revegetation practices compatible with those outlined in the document *A Guide for Minimizing the Impact of Pipeline Construction On the Native Prairie Ecosystem* (Alberta Environment, and Alberta Sustainable Resource Development, 2003), as amended, shall be used. Material handling and storage times in aid of native species regeneration are advised. Pipelines shall be constructed in a manner such that the surface disturbance on native prairie, and the degradation or loss of adjacent, undisturbed native prairie is minimized.

3.0 Wastewater Systems - Performance Standards

3.1 Treated Effluent Disposal to Surface Waters

3.1.1 Treated Effluent Quality

The treated effluent quality for a wastewater treatment facility shall be based on the more stringent of the quality resulting from the "Best Practicable Technology" (section 3.1.2) or the quality required based on receiving water assessments (section 3.1.3).

Exceptions to this rule are:

- the seasonal discharges to a receiving watercourse from wastewater lagoons designed and operated in accordance with AENV standards (section 4.2.1). No receiving water assessments are required for such releases; and

- when a water quality based limit is not technically attainable. In this case, an advanced technology limit may be adopted as an interim effluent limit.

3.1.2 Best Practicable Technology Standards

Only those technologies identified in Tables 3.1 and 3.2 are considered 'Best Practicable Technologies', and the corresponding effluent standards as 'Best Practicable Technology Standards'.

3.1.3 Receiving Water Quality Based Standards

Receiving water quality based standards shall be derived by calculating the maximum amount of substances that can be discharged under worst-case conditions while still maintaining instream water quality guidelines.

Detailed procedure for determining the receiving water quality based standards is included in the "Water Quality Based Effluent Limits Procedures Manual" published by AENV.

3.1.4 Disposal Criteria

Continuous discharge of effluent from treatment plants to a receiving watercourse shall be permitted if the recorded minimum mean monthly watercourse flow is ten times the total average daily discharge of treated effluent, and receiving water assessment indicates that there are no appreciable water quality impacts. However, if it can be demonstrated with a high level of certainty that no appreciable water quality impacts are projected to occur at 10:1 dilution, then discharge may be permitted at less than 10:1 dilution. Alternative methods of disposal and/or effluent storage facilities may be required if these conditions cannot be met.

Seasonal discharge of effluent from treatment plants, other than wastewater lagoons, to a receiving watercourse shall be reviewed on a site-specific basis; duration and timing of discharges will be determined based on receiving water assessment.

Continuous or seasonal discharges of effluent to lakes or other stagnant water bodies are generally discouraged. Such releases shall be reviewed on a site-specific basis, and will be permitted only if there are no water quality impacts. Water quality impacts will be assessed based on the anti-degradation policy, "Municipal Effluent Limits - Policy and Overview" in the Municipal Policies and Procedures Manual.

TABLE 3.1
BEST PRACTICABLE TECHNOLOGY STANDARDS FOR MUNICIPALITIES WITH CURRENT POPULATION <20,000

Type	Parameter	Standard	Sample	Comments
Secondary (mechanical)	CBOD TSS	25 mg/L 25 mg/L	Composite Composite	Monthly average of daily samples Monthly average of daily samples
Aerated lagoons	CBOD	25 mg/L	Grab	Monthly average of weekly samples
Wastewater lagoons 2 or 4 anaerobic cells (2 day retention time in each cell) 1 facultative cell (2 month retention time) 1 storage cell (12 month retention time)	None defined	None defined	None defined	Lagoons built to the specified design configuration and drained once a year between late spring and fall do not have a specified effluent quality standard. Early spring discharges may be allowed under exceptional circumstances to comply with any local conditions. Discharge period should not exceed three weeks unless local conditions preclude this rate of discharge.

Note:

1. Current population for municipalities served by the system shall be determined by taking into consideration the equivalent population for industrial waste discharges into the system. If site-specific information is not available, then equivalent population for industrial wastes shall be based on 70 g CBOD per person per day.
2. Sampling frequencies are based on continuous discharge of effluent to a body of water.
3. See Table 5.2 for the basis for selecting either 0 or 2 or 4 anaerobic cells in wastewater lagoons.

TABLE 3.2
BEST PRACTICABLE TECHNOLOGY STANDARDS FOR MUNICIPALITIES WITH CURRENT POPULATION >20,000

Type	Parameter	Standard	Sample	Comments
Tertiary (mechanical)	CBOD	20 mg/L	Composite	Monthly average of daily samples
	TSS	20 mg/L	Composite	Monthly average of daily samples
	TP	1 mg/L	Composite	Monthly average of daily samples
	NH_3-N	-	Composite	Need assessed on a site specific basis
	Total Coliform	1000/100 mL	Grab	Geometric mean of daily samples in a calendar month
	Fecal Coliform	200/100 mL	Grab	Geometric mean of daily samples in a calendar month
Aerated lagoons	CBOD	20 mg/L	Grab	Monthly average of weekly samples
	TP	1 mg/L	Grab	Monthly average of weekly samples
	NH_3-N	-	Grab	Need assessed on a site specific basis
	Total Coliform	1000/100 mL	Grab	Geometric mean of weekly samples in a calendar month
	Fecal Coliform	200/100 mL	Grab	Geometric mean of weekly samples in a calendar month
Wastewater lagoons 2 or 4 anaerobic cells (2 day retention time in each cell) 1 facultative cell (2 month retention time) 1 storage cell (12 month retention time)	None defined	None defined	None defined	Lagoons built to the specified design configuration and drained once a year between late spring and fall do not have a specified effluent quality standard. Early spring discharges may be allowed under exceptional circumstances to comply with any local conditions. Discharge period should not exceed three weeks unless local conditions preclude this rate of discharge.

Note:

1. Current population for municipalities served by the system shall be determined by taking into consideration the equivalent population for industrial waste discharges into the system. If site-specific information is not available, then equivalent population for industrial wastes shall be based on 70 g CBOD per person per day.
2. Sampling frequencies are based on continuous discharge of effluent to a body of water.
3. Facilities producing effluent with nitrogenous oxygen demand may be required to monitor for TBOD. The need for TBOD monitoring, and subsequent limit on NH_3-N will be assessed on a site-specific basis.
4. Bacteriological quality standards for total coliforms may be relaxed, if the owner demonstrates with some certainty that the wastewater being considered for disinfection is not typical of other municipal wastewaters.
5. Any sample yielding more than 400 total coliforms/100 mL shall be further investigated. Minimum action should consist of immediate re-sampling of the site.
6. Frequency of sampling for total and fecal coliforms may be reduced if it can be demonstrated with some certainty that bacteriological quality of effluent is consistent and the possibility of variance is minimal.
7. See Table 5.2 for the basis for selecting either 0 or 2 or 4 anaerobic cells in wastewater lagoons.

3.2 Treated Effluent Disposal to Land

3.2.1 Wastewater Irrigation

3.2.1.1 Minimum Treatment Requirement

If wastewater irrigation is chosen as the only method for the disposal of treated effluent, the minimum wastewater treatment shall be as follows:

1. Primary treatment (anaerobic cells in series or a facultative cell) followed by at least seven month storage; or
2. Secondary treatment with or without storage.

3.2.1.2 Treated Effluent Quality Standards

The treated effluent quality for wastewater irrigation shall meet the standards specified in Table 3.3.

TABLE 3.3
TREATED EFFLUENT QUALITY STANDARDS FOR WASTEWATER IRRIGATION

Parameter	Standard	Type of Sample	Comments
Total Coliform*	<1000/100 mL	Grab	Geometric mean of weekly samples (if storage is provided as part of the treatment) or daily samples (if storage is not provided), in a calendar month
Fecal Coliform*	<200/100 mL	Grab	Geometric mean of weekly samples (if storage is provided as part of the treatment) or daily samples (if storage is not provided), in a calendar month
CBOD	<100 mg/L	Grab/composite**	Samples collected twice annually prior to and on completion of a major application event
COD	<150 mg/L	Grab/composite**	Samples collected twice annually prior to and on completion of a major application event
TSS	<100 mg/L	Grab/composite**	Samples collected twice annually prior to and on completion of a major application event
EC	<2.5 ds/m	Grab/composite**	Samples collected twice annually prior to and on completion of a major application event
SAR	<9	Grab/composite**	Samples collected twice annually prior to and on completion of a major application event
pH	6.5 to 9.5	Grab/composite**	Samples collected twice annually prior to and on completion of a major application event

* For golf courses and parks only.

** Grab sample would suffice if storage is provided; Composite sample is required if storage is not provided.

3.2.2 Rapid Infiltration

3.2.2.1 Minimum Treatment Requirement

For rapid infiltration, a minimum of primary treatment shall be provided.

The system shall be designed in accordance with the joint Alberta Environmental Protection - City of Red Deer publication entitled Rapid Infiltration - A Design Manual.

3.2.3 Wetlands Disposal

3.2.3.1 Minimum Treatment Requirement

For wetlands disposal, a minimum of secondary or tertiary treatment shall be provided and the effluent quality shall meet the standards specified in tables 3.1 and 3.2.

Wetlands shall be evaluated and designed in accordance with Alberta Environmental Protection publication entitled Guidelines for the Approval and Design of Natural and Constructed Treatment Wetlands for Water Quality Improvement.

4.0 Wastewater Systems - Design Standards

4.1 Wastewater Collection

4.1.1 Sanitary Sewers

4.1.1.1 Minimum Pipe Diameter

Minimum pipe diameter for gravity sewer, in general, shall be 200 mm in diameter. However, under limited circumstances, sewers of not less than 150 mm diameter may be allowed if the owner can demonstrate that a 150 mm diameter sewer is adequate and will not be detrimental to the operation and maintenance of the sewer system.

The hydraulic capacity of a gravity sewer should be based on consideration of factors such as projected in-service roughness coefficient, slope, pipe material and actual in-service flows. In general, sewers larger than the minimum size required shall be chosen so that the minimum velocity at the average flow is not less than 0.6 m/s for self cleansing purposes, and the maximum velocity at the peak design flow is not greater than 3.0 m/s to minimize turbulence and erosion. Under exceptional circumstances, where velocities greater than 3.0 m/s are attained, provision shall be made to protect against displacement by erosion and impact.

For small diameter low pressure or vacuum sewer collection systems, the designer shall provide hydraulic calculations and/or supporting information to verify the proposal.

4.1.1.2 Minimum Pipe Slope

All gravity sewers between manholes shall be laid with uniform slopes equal to or greater than the minimum slopes outlined in Table 4.1.

These minimum slopes are not based on an assumed specific pipe roughness coefficient, but rather on historical satisfactory operation of sewers meeting or exceeding these slopes under varying flow conditions. Under special conditions, slopes slightly less than those shown may be permitted. If the proposed slope is less than the minimum slope of the smallest pipe which can accommodate the peak wastewater design flow, the actual depths and velocities at average, and peak wastewater design flow for each design section of the sewer shall be calculated by the designer and submitted to AENV. (See Sections 4.1.1.3 and 4.1.1.4).

For new construction, the pipe slope shall be determined using the minimum pipe diameter necessary for the design volume of wastewater. Further, a manhole outlet pipe diameter shall not be reduced to be smaller than the inlet pipe diameter to compensate for increased slope in the outlet line. In retrofit situations, where the minimum slope cannot be achieved due to site constraints, lower than the minimum slopes may be allowed. Under this situation, the owners shall make a commitment to undertake additional operation and maintenance measures to prevent solids deposition in the line.

4.1.1.3 Minimum Flow Depths

As per Section 4.1.1.2, slopes slightly less than those recommended for the 0.6 m/s velocity, when flowing full, may be permitted. Such decreased slopes will only be considered where the depth of flow will be 0.3 of the diameter or greater for average design flow. The owner of the sewer system shall give written assurance to AENV that any additional sewer maintenance required as a result of reduced slopes will be provided.

4.1.1.4 Solids Deposition

The pipe diameter and slope shall be selected to obtain practical velocities to minimize settling problems. As per Section 4.1.1.2, oversize sewers will not be approved on new constructions, to justify using flatter slopes. If the proposed slope is less than the minimum slope of the smallest pipe which can accommodate the peak wastewater design flow, the actual depths and velocities at average, and peak wastewater design flow for each design selection of the sewer shall be calculated by the designer and submitted to AENV.

4.1.1.5 Alignment

Curvilinear alignment of sewers may be allowed in the design of the collection system. Where curved sewers are used, the designer shall not exceed the maximum angle at which the joints remain tight. Curved sewers shall be laid with a radius of at least 60 m unless otherwise supported by manufacturer's specifications.

The minimum slopes for curved sewers shall be 50 percent greater than the minimum slopes required for straight runs; this requirement will be waived if the designer submits calculations to demonstrate that increased slope is not required to achieve self-cleansing velocity.

4.1.1.6 Frost Protection

Frost protection criteria for sewers is the same as for water mains (see section 1.9.2.1 for details).

4.1.1.7 Cross Connections

Cross connection prevention is the same as for water supply (see Section 1.9.3 for details).

TABLE 4.1
MINIMUM DESIGN SLOPES FOR SANITARY SEWERS

Sewer Diameter (mm)	Minimum Design Slope (m/100 m)
200	0.40
250	0.28
300	0.22
375	0.15
450	0.12
525	0.10
600	0.08

4.1.2 Wastewater Pump Stations

4.1.2.1 Site Constraints

Wastewater pump station structures and electrical and mechanical equipment should be protected from physical damage by the 100-year flood. Wastewater pump stations should remain fully operational and accessible during the 25-year flood.

4.1.2.2 Pumps

Wastewater pump station shall be designed with multiple pump units. Where only two units are provided, they shall be of the same size. Units shall have capacity such that, with any unit out of service, the remaining units will have capacity to handle the peak wastewater design flow.

Safety Ventilation

1. General

Adequate ventilation shall be provided for all pump stations. Ventilation systems, including fresh air intake louvers/openings shall be designed to function year round; screen openings should be sized to avoid build-up of frost during winter to prevent subsequent blockages. Where the dry well is below the ground surface, mechanical ventilation is required. If screens or mechanical equipment requiring maintenance or inspection are located in the wet well, permanently installed ventilation is required. There shall be no interconnection between the wet well and dry well ventilation systems.

Where ventilation of pump stations results in odour problems/complaints, the owner shall take steps to control the odour in accordance with section 5.3.3.

2. Air Inlets and Outlets

In dry wells over 5.0 m deep, multiple inlets and outlets are desirable. Dampers should not be used on exhaust or fresh air ducts and fine screens or other obstructions in air ducts should be avoided to prevent clogging.

In dry wells under 5.0 m deep, the fresh air shall be forced into the well at a point 150 mm above the pump floor and allowed to escape through vents in the roof; in wet wells, the fresh air shall be forced into the well at a point 150 mm above the high water level and allowed to escape through vents to the atmosphere.

3. Electrical Controls

Switches for operation of ventilation equipment shall be marked and located conveniently. All intermittently operated ventilation equipment shall be interconnected with the respective pit lighting system. Consideration should be given also to automatic controls where intermittent operation is used. The manual lighting ventilation switch shall override the automatic controls.

4. Fans, Heating and Dehumidification

The fan wheel shall be fabricated from non-sparking material. Automatic heating and dehumidification equipment shall be provided in all dry wells.

5. Wet Wells

Wet well ventilation may be either continuous or intermittent. Ventilation, if continuous, shall provide at least six complete air changes per hour; if intermittent, at least 30 complete air changes per hour during the period the wet well is occupied.

Air shall be forced into the wet well by mechanical means. For continuous ventilation, to facilitate free movement of air, the wet well may be exhausted at the highest elevation level in the structure at a rate not exceeding three air changes per hour; this rate shall not be exceeded to maintain positive pressure in the well.

Portable ventilation equipment shall be provided for use at submersible pump stations and wet wells with no permanently installed ventilation equipment.

6. Dry Wells

Dry well ventilation may be either continuous or intermittent. Ventilation, if continuous, shall provide at least six complete air changes per hour; if intermittent, at least 30 complete air changes per hour for ten minutes before entering the dry well and six complete air changes per hour to conserve heat during the period the dry well is occupied. A system of two-speed ventilation with an initial ventilation rate of 30 changes per hour for 10 minutes and automatic switch over to six changes per hour may be used.

7. Monitoring

Provision should be made in the system design to verify that the ventilation fan is operational, and that air change capacity is achieved. Portable or built-in sensing equipment for measurement of hydrogen sulphide, oxygen depletion, and combustibles should also be provided.

4.1.2.3 Separation

Dry wells, including their superstructure, shall be completely separated from the wet well. Common walls must be gas tight. All penetrations, e.g. electrical conduits, through the common wall shall be sealed.

4.1.2.4 Access

Suitable and safe means of access to dry wells and to wet wells shall be provided for inspection and cleaning. The access into the wet well shall not be through the superstructure where the pumping equipment and appurtenance may be housed. Gasketed replacement plates shall be provided to cover the opening to the wet well for pump units removed for servicing. To minimize the need for entry to the wet well, valving shall not be located in the wet well.

4.1.2.5 Water Supply

There shall be no physical connection between any potable water supply and a wastewater pumping station which under any conditions might cause contamination of the potable water supply. If a potable water supply is brought to the station, it should comply with conditions stipulated under Section 4.2.3.4 (3). In-line backflow preventers are not acceptable.

4.1.3 Screens and Grit Removal Facilities

4.1.3.1 Access and Ventilation

Screens located in pits more than 1.2 m deep shall be provided with stairway access. Access ladders, in lieu of stairways, are acceptable for pits less than 1.2 m deep.

Screening devices, installed in a building where other equipment or offices are located, shall be:

- isolated from the rest of the building,
- provided with separate outside entrances, and
- provided with separate and independent fresh air supply.

Fresh air shall be forced into enclosed screening device areas or into open pits more than 1.2 m deep. Dampers should not be used on exhaust or fresh air ducts and fine screens or other obstructions should be avoided to prevent clogging. Where continuous ventilation is required, at least six complete air changes per hour shall be provided. Where continuous ventilation would cause excessive heat loss, intermittent ventilation of at least 30 complete air changes per hour shall be provided when workmen enter the area.

Switches for operation of ventilation equipment should be marked and located conveniently. All intermittently operated ventilation equipment shall be interconnected with the respective pit lighting system. The fan wheel should be fabricated from non-sparking material. Gas detectors shall also be provided.

For grit removal facilities housed above or below grade, ventilation shall be provided either continuously at a rate of six complete air changes per hour, or intermittently at a rate of 30 air changes per hour.

4.1.4 Health and Safety Act

The design and construction of all components of the wastewater collection system shall conform to the safety provisions of the *Alberta Occupational Health and Safety Act, Regulation and Code*.

4.2 Wastewater Treatment and Disposal

4.2.1 Wastewater Lagoons

4.2.1.1 General Requirements

The minimum design standards for wastewater lagoons are a function of the average daily design flow. AENV has established acceptable cell configurations based on design flow; the following sections outline the respective component configurations required to meet the minimum standards for wastewater lagoons as specified in Section 3.0 Performance Standards - Wastewater Systems.

4.2.1.2 System Components and Configuration

1. **Anaerobic Cells**

   Anaerobic cells shall operate at a minimum depth of 3.0 m and retain influent flow for a 2-day period based on average daily design flow.

   In order to provide a cell bottom of at least three (3) metres square for adequate mobility of construction equipment and sufficient bottom area for sludge accumulation, the minimum practical design volume of an anaerobic cell with 3:1 inside slopes and 3 m of operating depth is approximately 500 m^3. Therefore, the minimum average daily design flow at which an anaerobic cell is practical is approximately 250 m^3/d.

   When design flows warrant the provision of anaerobic cells, there shall be a minimum of two and a maximum of four cells operated in series. The number of anaerobic cells is a function of design daily flows as illustrated in Table 4.2 and Figure 4.1.

   In designing the anaerobic cells, consideration shall be given to incorporate desludging features.

2. **Facultative Cells**

   The facultative cell shall operate at a maximum depth of 1.5 m and retain influent wastewater for at least 60 days based on average daily design flow. The purpose of this cell is to biologically stabilize the wastewater under predominantly aerobic conditions. The cell(s) follows the anaerobic treatment (in cases where anaerobic cells are warranted), precedes long-term detention storage, and is a requirement for all wastewater lagoon systems in Alberta. Refer to Figure 4.1.

TABLE 4.2
WASTEWATER LAGOON REQUIREMENTS

Average Daily Design Flow (m^3)	Number of Anaerobic Cells	Requirement for Facultative Cell(s)	Requirement for 12 month Storage Cell(s)
Less than 250	0 Min. depth = 3.0 m	Yes Max. depth = 1.5 m	Yes Max. depth = 3.0 m
250-500	2 Min. depth = 3.0 m	Yes Max. depth = 1.5 m	Yes Max. depth = 3.0 m
Greater than 500	4 Min. depth = 3.0 m	Yes Max. depth = 1.5 m	Yes Max. depth = 3.0 m

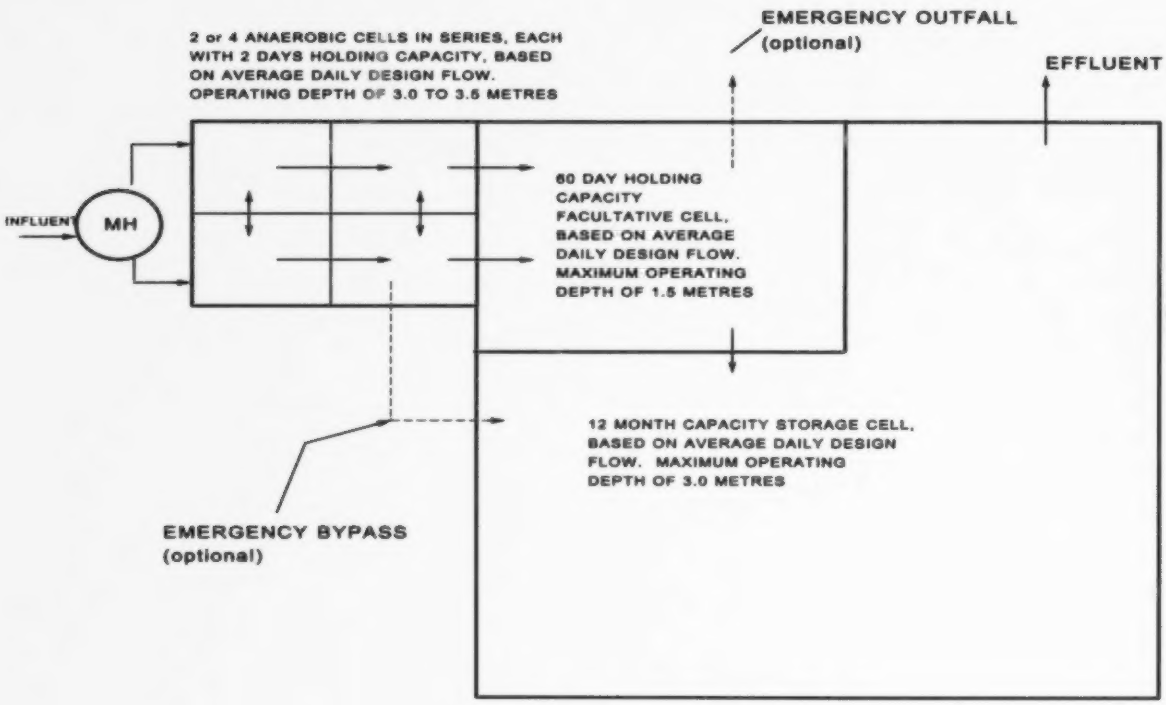
3. Storage Cell(s)

The storage cell(s) shall operate at a maximum depth of 3.0 m and shall retain influent wastewater for a minimum retention period of 12 months based on average daily design flows. However, if historical climatology data shows that average evaporation exceeds the average precipitation, then the net evaporation may be taken into account in sizing the storage cells. Under this scenario, the minimum "free-board" shall be increased to accommodate additional flows in an anomalous precipitation year.

The purpose of storage is to provide additional wastewater treatment (including nutrient removal) under facultative conditions, and to reduce the environmental impact on the receiving drainage course by facilitating the annual discharge of high quality effluent wastewater.

FIGURE 4.1 .1

TYPICAL WASTEWATER LAGOONS


EMERGENCY OUTFALL
(optional)
2 or 4 ANAEROBIC CELLS IN SERIES, EACH WITH 2 DAYS HOLDING CAPACITY, BASED ON AVERAGE DAILY DESIGN FLOW. OPERATING DEPTH OF 3.0 TO 3.5 METRES
EFFLUENT
INFLUENT
MH
60 DAY HOLDING CAPACITY FACULTATIVE CELL, BASED ON AVERAGE DAILY DESIGN FLOW. MAXIMUM OPERATING DEPTH OF 1.5 METRES
12 MONTH CAPACITY STORAGE CELL, BASED ON AVERAGE DAILY DESIGN FLOW. MAXIMUM OPERATING DEPTH OF 3.0 METRES
EMERGENCY BYPASS
(optional)

4.2.1.3 Design Considerations

There are a number of siting and design features which must be considered in order to protect pubic health, to maximize public safety, and also to meet the minimum standards for wastewater lagoon design.

1. **Isolation of Cells**

   For maintenance purposes, the cells may have to be by-passed for brief periods of time. The design shall take this into account and have provisions built-in to isolate any cell.

2. **Setback Distances**

   Setback distances from wastewater lagoons are required to buffer the effect of potential odours and to provide a margin of public safety. Setbacks also serve to protect the physical integrity of nearby buildings and roads.

   Table 4.3 outlines the minimum horizontal setback distances for wastewater lagoons.

3. **Site Constraints**

   The facility shall be protected from physical damage by the one hundred year flood. Treatment works should remain fully operational and accessible during the twenty-five year flood. On a site-specific basis, it may also be necessary to provide an adequate setback distance to an adjacent drainage course.

4. **Fencing**

   The wastewater lagoons shall be enclosed with a fence. The fence shall be designed and constructed to preclude the entrance of children and to discourage trespassing. The fence shall also serve to preclude the entrance of livestock. Where the lagoons are located near developed areas, a chain link fence may be required to preclude children from gaining entry.

   Fences shall be located away from the outside toe of the berms in order to facilitate mowing and maintenance operations. In addition, an access gate shall be provided to allow entry of maintenance equipment, and this gate shall be provided with a lock to preclude entrance of unauthorized personnel.

5. **Signs**

   Warning signs shall be provided at appropriate locations along the fenced perimeter of the wastewater lagoons. Each sign should identify the facility (by municipality or owner), advise against trespassing, and provide emergency contact phone numbers and/or addresses.

6. **Access**

   All-weather access to the wastewater lagoons shall be provided.

7. Surface Runoff

Wastewater lagoons shall have adequate site drainage to divert surface runoff which would otherwise cause damage to the system.

TABLE 4.3
SETBACK DISTANCES FROM WASTEWATER LAGOONS

Minimum setback distance (m) from the "working area" of the wastewater lagoon to:	
The property line of the land where the lagoon is located	30
The designated right-of-way of a rural road or railway	30
The designated right-of-way of a primary or secondary highway	100
An "Occupied Building" where the lagoon serves a designated municipality	300
Any "Occupied Building" on the property of a privately owned rural development which the lagoon serves	300

* "Occupied building" means a building within which one or more persons reside, work or are served for four or more hours a day; and two or more days a week; and eight or more weeks a year. Without limiting the generality of the foregoing, this includes such developments as school, hospital, food establishment, residences, etc.

** "Working area," means, those areas of a parcel of land that are currently being used or will be used for the processing of wastewater.

4.2.1.4 Seepage Control

The following sections are a summary of the Alberta Environment publication entitled "Design and Construction of Liners for Municipal Wastewater Stabilization Ponds in Alberta". To obtain additional detail, reference should be made to this publication.

1. Control Criteria

The control of seepage from wastewater lagoons is one of the most important aspects of pond design, construction, operation and maintenance. The maximum allowable hydraulic conductivity for pond liners consisting of in-situ material, compacted clay, bentonite and sand, asphalt concrete or other porous materials in which seepage is:

$$Maximum\ K_T = \frac{C\ x\ T}{2 + T}$$

governed by Darcy's Law, shall be calculated using the following equation:

where:

K_T = maximum hydraulic conductivity of liner in the field, being at least one order of magnitude greater than the laboratory value, m/s
T = required or proposed thickness of liner, m
C = 5.2×10^{-9} m/s.

2. Site Selection and Investigations

Potential sites for wastewater lagoons shall be critically evaluated in terms of their ability to meet the requirements for seepage control. Preliminary site selection should be based on existing and pertinent topographical, geological, hydrogeological and geotechnical data. The most suitable site(s) based on these considerations shall then be investigated in detail.

The detailed site investigation shall include a review of groundwater use in the area, a reconnaissance of surface features (such as bedrock outcrops and drainage courses) and a subsurface exploration consisting of a minimum of either five (5) boreholes or one (1) borehole per 2 ha of cell area, whichever is greater. The number of boreholes drilled will depend on the complexity of the geology determined by the initial drilling program. Additional boreholes would be required to delineate a site with features such as sand layers or large sand lenses. Each borehole should be drilled to a depth of 6 m below the proposed invert elevation and at least one borehole should be drilled to a depth of either 20 m or to auger refusal in bedrock, whichever occurs first. At least three boreholes should penetrate the groundwater table in order to determine groundwater conditions including water-table elevations, flow direction and gradient. In situations where a shallow aquifer has been identified as part of the site reconnaissance or drilling programs, nests of wells should be installed to evaluate vertical groundwater flow conditions. Each borehole should be logged and soil samples taken; upon completion of the site investigation each borehole not developed as a monitor well shall also be properly sealed. In-situ and laboratory soil tests as outlined in the Manual for "Design and Construction of Liners for Municipal Wastewater Stabilization Ponds in Alberta" should be conducted.

Monitor wells would be used to measure water levels and, once the levels have stabilized, permeability tests can be conducted to determine the hydraulic conductivity of the material in which the monitor well is completed. The wells should be surveyed with respect to elevation and location in order to determine groundwater elevation and the direction of groundwater flow.

Sites in flood plains, sites located above buried channel aquifers, or sites having either hilly terrain, high bedrock, fissured rock formations or high water tables should be avoided to protect against possible groundwater pollution problems and to prevent adjacent lands from being adversely affected by seepage. Areas of high water table should be avoided due to possible construction /design problems. The bedrock surface should be a minimum of 3.0 m below the invert of any cell.

3. Liner Design

The following design requirements shall be used when designing and constructing waste stabilization pond liners:

(i) Natural in-situ liners shall have a minimum thickness of 0.9 m below the entire bottom, shall be relatively uniform, and shall be completely free of hydrogeologic windows such as sand and silt. Engineered side slope liners shall be provided if the horizontal hydraulic conductivity of the in-situ liner does not meet the seepage control criterion or if berms are constructed with fill material;

(ii) Compacted Clay Liners shall have a minimum thickness of 0.6 m on the bottom and 1.2 m on the side slope (measured perpendicular to the slope). The liner should be constructed in 150 to 200 mm lifts and compacted to the required density, within the required moisture content range, to achieve the required seepage control criterion. When determining whether the seepage control criterion is met, the actual hydraulic conductivity of the liner in the field shall be assumed to be at least one order of magnitude greater than laboratory values;

(iii) Bentonite and Sand Admix Liners. Bentonite shall only be considered for lagoon liners where mixing with native sands or silts allows a uniform bentonite and sand/silt admix. Only moderate to high swelling sodium bentonite shall be used. The bentonite application rate required to meet the seepage control criterion should be determined by laboratory permeability tests and then increased by 25% to allow for field conditions. The admix liner shall be at least 100 mm thick after compaction, and any portion of the liner which is susceptible to weathering when exposed shall be covered with suitable soil material;

(iv) Asphalt Liners. The only asphalt liners recommended for use in wastewater lagoons are spray-on bitumen over hydraulic asphalt concrete and spray-on bitumen over soil asphalt. Hydraulic asphalt concrete liners shall be a minimum of 100 mm thick comprising two 50 mm lifts with staggered joints. Soils asphalt liners should be mixed with 150 mm of native sandy soil. In all cases asphalt shall be placed over non-frost susceptible base courses or sub grades. The spray-on bitumen covering the hydraulic asphalt concrete or soil asphalt surface should provide a uniform 2 mm thick membrane; and

(v) Flexible Polymeric Membrane Liners. The minimum thickness of membranes used to line wastewater lagoons is 5×10^{-3}mm (20 mils). Membranes less than 15×10^{-3}mm (60 mils) thick shall be covered with a 300 mm layer of fine-grained soil on the pond side slopes to prevent liner damage. PVC and other membrane liner materials that are susceptible to weathering when exposed shall be covered with soil on both the side slopes and bottom. A stable and well prepared subgrade and proper membrane installation (with particular emphasis on seaming) is necessary for successful performance of the liner. A system for venting gas generation beneath the liner should be considered.

4. Groundwater Monitoring

A post-construction groundwater monitoring program is required to assess the performance of the liner.

The monitor wells installed during the detailed site investigation could be used for groundwater monitoring after completion of the ponds and associated facilities. If some or all of the wells have been lost during construction, new monitor wells should be installed where required.

Monitor wells should be constructed of at least schedule 40 PVC pipe that is a minimum of 50 mm in diameter. The screen should be of the same materials and should be machine slotted. Pipe connections should be threaded rather than glued to prevent the introduction of contaminants into the well. A sand or gravel pack must be placed in the annulus around the screened section and should extend not more than half a metre above the screen. The remaining annulus should be sealed to surface with bentonite or a cement slurry. The portion of the pipe sticking up above ground should have a metal protector or other appropriate barrier system installed around it. Wells should be provided with lockable caps to prevent tampering.

Monitor wells shall be located close to the toe of the perimeter berms. The depth of completion will be governed by the information obtained during the site investigation phase. It is important, however, that wells intersect the water table. A minimum of four wells shall be placed around the pond with the well spacing ideally not to exceed 100 metres.

The following groundwater monitoring program shall be instituted for all new wastewater lagoons:

- Wells shall be analyzed for physical, chemical and biological parameters four times in each quarter of the first year of operation. This will provide the baseline data to be compared with future analysis.
- The first analysis from each monitoring well shall be undertaken prior to putting a new lagoon into operation. The following three analyses shall be carried out approximately three months apart to cover all seasons in the year.
- Water samples shall be analyzed for routine water chemistry, total Kjeldahl nitrogen and chemical oxygen demand.
- Wells shall be monitored for water levels whenever a sample is collected for chemical analysis, and also during the lagoon discharge period. During the lagoon drainage period, one set of readings shall be taken:
    i. immediately before discharge,
    ii. immediately after discharge, and
    iii. approximately one month after the end of the discharge period.

4.2.1.5 Construction Features

There are a number of design and construction features which should be followed in order to facilitate good operation and maintenance of the wastewater lagoons.

1. Berms

(i) Embankment tops shall have a minimum width of 3 m to provide a driving surface for maintenance vehicles;

(ii) Embankment slopes shall be as steep as the safe operation of equipment will permit and the local soil condition will allow. Slopes for (a) the outside of the berm of 4:1 to 5:1 (horizontal to vertical), and (b) the inside of the berm of 3:1 or less, are recommended;

(iii) Inside embankment slopes on the windward side of the prevailing wind shall be armoured with rip-rap or other suitable material. The required size of riprap may be greater where large cells are constructed in high wind areas;

(iv) The "freeboard", the vertical distance between the high water level and the top of the berm, shall be a minimum of 0.6 m to allow for fluctuation of the operating high water level in the cell. Increased 'freeboard' may be required where high winds and steep embankments result in water scouring or to accommodate additional flows in an anomalous precipitation year when evaporation losses are taken into account in sizing the storage cells; and

(v) Special soils conditions may require the berm(s) to be "keyed" into the subsoil in order to preclude the horizontal seepage across the base of the berm. Determination of whether the "keying in" or "cut-off" procedure is appropriate for berm construction will be based on a geotechnical evaluation and engineering soils report.

2. Inlet Structures

Control manholes are commonly used for access to inlet piping and to regulate the flow from the influent pipeline to the anaerobic or facultative cells. The invert of the influent pipeline entering the control manhole should be above the maximum operating level of the anaerobic or facultative cell. The inlet pipe from the manhole to the anaerobic or facultative cell should enter the cell at approximately 1/3 to 2/3 the cell depth.

In the design of inlet structures, consideration should also be given to receive truck-haul sewage from holding tanks.

3. Outlet and Drain Structure

The drain from the final storage cell shall be installed to ensure that a minimum of 150 mm of liquid is retained. A manhole shall be provided to house a valve and vertical overflow pipe for the drainage pipe, and the drainage valve shall be equipped with a long stem so that it can be operated without entering the manhole. Erosion protection shall also be provided at the location of effluent discharge.

The drain from the final storage cell shall have capacity to ensure that the annual discharge of final effluent is completed in a period of three weeks or less.

4. Flow Measurement

Section 5.2.4.2 (5) outlines the requirements for wastewater flow measurement at pumping stations. In cases where the system is a total gravity system, i.e. no pumping station required to transport flows to the wastewater lagoons, and the system has a capacity greater than 500 m^3/d, a portable or permanent flow measuring device shall be provided at the inlet of the wastewater lagoons.

4.2.2 Wastewater Evaporation Lagoons

Where conventional wastewater lagoons or mechanical plant are impractical because of effluent discharge restrictions, it may be necessary to consider the provision of wastewater evaporation lagoons.

4.2.2.1 System Components and Configuration

For systems with average daily design flows of less than 250 m^3, the system may be designed with one evaporation cell. Provision must be made at the inlet to the cell for settlement and removal of sludge. For systems with flows larger than 250 m^3/d, the evaporation cell shall be preceded by two anaerobic or two facultative cells.

1. **Anaerobic Cells**

   Anaerobic cells shall operate at a minimum depth of 3.0 m and for each cell to retain influent flow for a two-day period based on average daily design flow.

2. **Facultative Cells**

   Facultative cells shall be designed with a maximum operating depth no greater than 1.5 m, and for each cell to retain influent flow for a thirty-day period based on average daily design flow.

3. **Evaporation Cells**

   When establishing the size of an evaporation lagoon, consideration shall be given to local meteorological conditions such as rainfall, evaporation, and evapotranspiration. These factors shall be combined with the design influent flows to calculate the required surface area of the cell(s). In no case shall an evaporation lagoon provide less than three years of storage capacity based on average daily design flows. In practice, more than three years of storage will be required in all but the most southern areas of Alberta. Evaporation cells shall have a depth of no greater than 1.5 m.

4.2.2.2 Design Considerations

Design requirements outlined for wastewater lagoons in Sections 4.2.1.3, 4.2.1.4 and 4.2.1.5 also apply to evaporation lagoons.

4.2.3 Mechanical Wastewater Treatment

4.2.3.1 General Requirements

Mechanical wastewater treatment plants shall be designed such that the treated effluent quality meets the performance standards stipulated under Section 3.1 if the effluent is discharged to surface waters, and the standards stipulated under Section 3.2 if the effluent is disposed to land.

4.2.3.2 Design Considerations

1. **Setback Distances**

   Setback distances from mechanical wastewater treatment plants are required to prevent the occurrences of objectionable odours in subdivision when plants are operated normally and within designed capacities. Table 4.4 outlines the minimum horizontal setback distances from mechanical plants including aerated lagoons.

2. **Others**

   Design features for site constraints, fencing, signs and access shall be in accordance with Sections 4.2.1.3 (3), (4), (5) and (6) respectively.

4.2.3.3 Safety

Adequate provision shall be made to effectively protect the operator and visitors from hazards. The following shall be provided to fulfil the particular needs of each plant:

1. Enclosure of the plant site with a fence and signs designed to discourage the entrance of unauthorized persons and animals;

2. Hand rails and guards around tanks, trenches, pits, stairwells, and other hazardous structures. Height and size of handrails to conform to Alberta Building Code standards. Materials to be non-corrosive;

3. Gratings over appropriate areas of treatment units where access for maintenance is required. Grating material to be non-corrosive (carbon steel not acceptable);

4. First aid equipment;

5. "No Smoking" signs in hazardous areas;

6. Protective clothing and equipment, such as self-contained breathing apparatus, gas detection equipment, goggles, gloves, hard hats, safety harnesses, etc.;

7. Portable blower and sufficient hose;

8. Portable lighting equipment;

9. Gas detectors;

10. Appropriately placed warning signs for slippery areas, non-potable water fixtures, low head clearance areas, open service manholes, hazardous chemical storage areas, flammable fuel storage areas, etc.;

11. Provision for confined space entry in accordance with Alberta Occupational Health and Safety Act and Regulations; and

12. Provision for anaerobic digesters and sludge holding tanks in accordance with the latest edition CAN/CGA - B105, Code for Digester Gas and Landfill Gas Installations.

TABLE 4.4
SETBACK DISTANCES FROM MECHANICAL TREATMENT PLANTS INCLUDING AERATED LAGOONS

Minimum setback distance (m) from the "working area" of the operating mechanical treatment plants or aerated lagoons to:	
The property line of the land where the operating mechanical treatment plants or aerated lagoons is located	30
The designated right-of-way of a rural road or railway	30
The designated right-or-way of a primary or secondary highway	100
An "Occupied Building" where the operating mechanical treatment plants or aerated lagoons serves a designated municipality	300
Any "Occupied Building" on the property of a privately owned rural development which the operating mechanical treatment plants or aerated lagoons serves	300

* "Occupied building" means a building within which one or more persons reside, work or are served for four or more hours a day; and two or more days a week; and eight or more weeks a year. Without limiting the generality of the foregoing, this includes such developments as school, hospital, food establishment, residences, etc.

** "Working area" means, those areas of a parcel of land that are currently being used or will be used for the processing of wastewater.

4.2.3.4 Water Supply and Sanitary Facilities

1. General

For mechanical wastewater treatment plants, an adequate supply of potable water under pressure should be provided for use in the laboratory and for general cleanliness around the plant. No piping or other connections shall exist in any part of the treatment works which, under any conditions, might cause the contamination of a potable water supply.

2. Direct Connections

Potable water from a municipal or separate supply may be used directly at points above grade for the following hot and cold supplies:

(i) lavatory;
(ii) water closet;
(iii) laboratory sink (with vacuum breaker);
(iv) shower;
(v) drinking fountain;
(vi) eye wash fountain; and
(vii) safety shower.

Hot water for any of the above units shall not be taken directly from a boiler used for supplying hot water to a sludge heat exchanger or digester-heating unit.

3. Indirect Connections

Where a potable water supply is to be used for any purpose in a plant other than those listed in subsection 2, a break tank, pressure pump, and pressure tank shall be provided. Water shall be discharged to the break tank through an air gap at least 150 mm above the maximum flood line or the spill line of the tank, whichever is higher.

A sign shall be permanently posted at every hose bib, faucet, hydrant, or sill cock located on the water system beyond the break tank to indicate that the water is not safe for drinking.

4. Separate Non-Potable Water Supply

Where a separate non-potable water supply is to be provided, a break tank will not be necessary, but all system outlets shall be posted with a permanent sign indicating the water is not safe for drinking.

5. Sanitary Facilities

Toilet, shower, lavatory, and locker facilities should be provided in sufficient numbers and convenient locations to serve the expected plant personnel.

4.2.4 Aerated Lagoons

4.2.4.1 General Requirements

Aerated lagoons shall be of the "completely mixed type". The design of completely mixed aerated lagoons requires enough oxygen transfer to satisfy the applied BOD loading and sufficient mixing to maintain a uniform solids concentration in the complete mix cells.

The system shall be designed such that the treated effluent quality meets the performance standards stipulated under Section 3.1 if the effluent is discharged to surface waters, and the standards stipulated under Section 3.2 if the effluent is disposed to land.

4.2.4.2 System Components and Configuration

Typical completely mixed aerated lagoons, treating domestic wastewater, shall consist of the following:

1. A completely mixed cell, having a total retention time of at least two days based on maximum monthly average daily design flow. Aeration equipment in this cell shall be designed to achieve complete mixing by maintaining a uniform solids concentration;

2. A minimum of two partially mixed aerated cells, having a total retention time of at least 28 days based on maximum monthly average daily design flow. The two aerated cells may operate in series or in parallel, with each cell sized to 50% of the maximum monthly average daily design flow; cells in series are preferred for continuous discharge of treated effluent. Aeration equipment in this cell shall be designed to maintain a total dissolved oxygen concentration of at least 2.0 mg/L during peak loading periods; and

3. A polishing cell having a minimum hydraulic retention of five days based on maximum monthly average daily design flow is required for continuous discharge systems.

Operating depths of 5 m are recommended for good mixing efficiency and to reduce heat loss during winter months.

The foregoing configuration and the retention times may be varied if the influent characteristics are considerably different from the typical characteristics of domestic wastewater (BOD - 200 mg/L, TSS - 200 mg/L). Any variation shall be substantiated by the municipality submitting the design calculations for the modified lagoon system.

4.2.4.3 Design Considerations

A number of the design requirements outlined for wastewater stabilization ponds and mechanical wastewater treatment plants also apply to aerated lagoon systems. These design features which shall be applied to aerated lagoons include:

1. Site constraints - 4.2.1.3 (2)
2. Fencing - 4.2.1.3 (3)
3. Signs - 4.2.1.3 (4)
4. Access - 4.2.1.3 (5)
5. Surface Runoff - 4.2.1.3 (6)

6. Seepage Control - 4.2.1.4 (1-4)
7. Construction Features- 4.2.1.5 (1-4)
8. Setback Distances - 4.2.3.2 (1)

4.2.5 Disinfection

4.2.5.1 General Requirements

Municipalities serving current population greater than 20,000 and discharging to surface waters shall provide disinfection of the effluent to meet the performance standards stipulated in Section 3.1. The need for wastewater disinfection for municipalities serving a current population less than 20,000 with continuous discharge, will be determined on a site specific basis, based on receiving water assessment. Municipalities disposing on land (golf courses, parks, wetlands) shall also provide disinfection of the effluent to meet the performance standards stipulated in Section 3.2. In general, where a public health hazard may be created by discharge of wastewater plant effluent, disinfection shall be required.

Ultraviolet irradiation, chlorine or chlorine derivatives are commonly approved and used for wastewater disinfectants in Alberta; alternatives include ozone, bromine, iodine, and gamma radiation. The choice of wastewater disinfectant should be based on a number of factors including flow rates, impact on the receiving stream, disinfectant application and demand rates, wastewater pH, the costs and availability of feed equipment, the costs and availability of specific chemicals, and the operation and maintenance factors.

When chlorine is used as the disinfectant, dechlorination of wastewater effluent is required to reduce the toxicity due to chlorine residuals. End of pipe limit for chlorine will be based on receiving water assessment.

4.2.6 Wastewater Treatment Chemicals

Chemicals selected for use in wastewater treatment plants must be such that they will not adversely affect the operation of the wastewater or sludge treatment processes and will not leave dangerous residuals in the effluent or sludge leaving the plant. The purity of chemicals proposed to be used should be determined. Waste streams from industry, such as ferrous chemicals, can be used provided that they are not contaminated with other hazardous materials.

See Section 2.3.10, "Water Treatment Chemicals" for information on Labels and Material Safety Data Sheets and Storage Handling.

4.2.7 Effluent Disposal

4.2.7.1 Disposal to Surface Waters

1. Seasonal Discharges

The drainage course receiving seasonal discharges from a wastewater treatment plant must be capable of transporting the effluent to the ultimate receiving watercourse without the occurrence of either flooding of adjacent lands or erosion of the drainage course itself.

Wastewater lagoons shall be drained once a year between late spring and fall; the discharge period should not exceed three weeks unless the local conditions preclude this rate of discharge. Early spring discharges may be allowed only under exceptional circumstances to comply with any local conditions.

Where drainage course improvements are required to handle a seasonal pond discharge, prior approval must be obtained under the Water Act in Alberta. In addition, the owner of the facility should obtain easements across privately owned land along the drainage course in areas where flooding problems are foreseen.

2. Continuous Discharges

The drainage course receiving discharges from a wastewater treatment plant must be capable of transporting the effluent to the ultimate receiving watercourse without the occurrence of either flooding of adjacent lands or erosion of the drainage course itself.

Piped systems are the preferred method of transporting continuous effluent discharge to an approved receiving watercourse. Under some conditions, however, it may be permissible to use a manmade or intermittent natural drainage course to convey the continuous discharge for a short distance to the receiving watercourse.

4.2.7.2 Disposal to Land

In Alberta, the most common method of land disposal of effluent is through wastewater irrigation, and to a lesser extent through wetlands. Rapid infiltration is generally uncommon, and, so far, has not been practiced by any municipality in Alberta.

1. Wastewater Irrigation

Minimum treatment requirement and treated effluent quality standards for wastewater irrigation are outlined in Section 3.2.1.

Wastewater irrigation, as an effluent disposal option, should be considered only when it is environmentally acceptable and agriculturally beneficial. This may be suitable in regions where additional moisture can be utilized for improved crop production. Application amounts should be based on the net consumptive use of the crop being grown while taking into account the seasonal moisture deficiencies, application efficiencies, and any additional needs as may relate to leaching requirements. The primary objective should be the enhancement of crop production. The root zone of productive soils can also serve as one of the most active media for the decomposition, immobilization, or utilization of municipal wastes. An added benefit of wastewater irrigation is that wastewater may be safely released to the environment at somewhat lower levels of treatment than would apply for other disposal options.

Wastewater suitability for irrigation purposes is based on specific water quality parameters. These parameters should be tested prior to and during their release. Site acceptability is to be based on pertinent soil and geologic properties, topography, hydrology, climate, and zoning and cropping intentions.

Land application of wastewater shall be in accordance with the latest edition of Guidelines for Municipal Wastewater Irrigation, published by AENV.

2. Rapid Infiltration

In the rapid infiltration process, wastewater is applied at high rates for a period from several days to several weeks, and allowed to infiltrate and percolate into the soil. A rest period is then required for the infiltration and treatment capacity of the soil to be restored. The process can be used for wastewater treatment where soil and geologic conditions provide rapid infiltration and high permeability. This process may have a deleterious effect on both groundwater and surrounding surface water.

Design of rapid infiltration (sometimes referred to as infiltration-percolation) systems shall be done in accordance with the joint Alberta Environment - City of Red Deer publication entitled Rapid Infiltration - A Design Manual. In general, the following shall apply:

(a) a minimum of primary treatment;

(b) a minimum of two wastewater storage cells to provide for periodic basin maintenance during the rest period;

(c) detailed multidisciplinary site investigations that include, but is not limited to soils, hydrology, hydrogeology, topography, and climate;

(d) subsurface groundwater flow definition; and

(e) subsurface drains, as required.

3. Wetlands Disposal

Disposal of effluent on wetlands shall be considered only if the wastewater has received secondary or tertiary treatment and the effluent quality meets the standards specified in Tables 3.1 and 3.2.

Wetlands shall be evaluated and designed in accordance with the Alberta Environmental Protection publication entitled Guidelines for the Approval and Design of Natural and Constructed Treatment Wetlands for Water Quality Improvement.

4.2.8 Biosolids (Sludge) Disposal

Biosolids are principally organic in nature, and contain varying quantities of metals, nutrients, salts, grit, synthetic organics and pathogenic organisms. The exact composition of biosolids is a function of the wastewater being treated and can vary significantly from municipality to municipality depending on the quantity and quality of industrial, commercial and institutional inputs. The quantity, quality and characteristics of biosolids from wastewater treatment may be altered by employing various treatment techniques (see Section 5.4).

In general, as treatment efficiency increases the volume of biosolids increases. Biosolids must be disposed of in an environmentally acceptable manner if wastewater treatment, as a whole, is to be considered effective. Commonly employed biosolids disposal methods include:

1. ocean dumping;
2. sanitary landfilling;
3. incineration;
4. permanent lagoon storage; and
5. land application.

Of these disposal methods, the only way of re-utilizing and conserving the useful constituents in biosolids is through its application to land. This method is also consistent with the objective of disposing of biosolids in an environmentally acceptable manner.

The application of biosolids to agricultural lands can benefit both municipalities and farmers. To municipalities land application is often the most practical method of biosolids disposal and to farmers biosolids is an excellent potential soil conditioner and fertilizer.

Land application of biosolids shall be carried out in accordance with the latest edition of Guidelines for the Application of Municipal Wastewater Sludges to Agricultural Lands published by AENV. The purpose of these guidelines is to:

1. discuss the risks and benefits associated with the use of biosolids on agricultural land; and

2. outline biosolids application rates and procedures that should be used when applying biosolids to agricultural land in Alberta.

In addition to being a potential soil conditioner and fertilizer, biosolids may also contain pathogenic organisms and quantities of elements and chemicals which can adversely affect crop quality and yield and result in pollution of surface and groundwaters. These guidelines represent an attempt to provide criteria which will maximize the benefits associated with land application of biosolids while minimizing the potential risks.

4.2.9 Health and Safety Act

The design and construction of all components of the wastewater system shall conform to the safety provisions of the *Alberta Occupational Health and Safety Act, Regulation and Code*.

5.0 Wastewater Systems - Design Guidelines

5.1 Design Criteria

5.1.1 Estimating Wastewater Flows

The following sections outline methodologies for quantifying wastewater flows. From a qualitative point of view, owners of wastewater systems are encouraged to develop and implement policies and programs to promote "at source reduction" for any and all contaminants in wastewater.

5.1.1.1 Residential (Population-Generated)

If no existing data exists, the peak (population-generated) flow for a residential population may be determined by the following formula:

$$Q_{PDW} = \frac{G \times P \times Pf}{86.4}$$

where: Q_{PDW} = the peak dry weather design flow rate (L/s)
G = the per capita average daily design flow (L/d)
P = the design contributing population in thousands
Pf = a "peaking factor".

The peaking factor (Pf) should be the larger of 2.5 or Harmon's Peaking Factor

where:

Harmon's Peaking Factor = $1 + 14/(4 + P^{1/2})$

where:
P = the design contributing population in thousands.

5.1.1.2 Commercial/Institutional and Industrial

1. Determination of Average Flow

For detailed system design, the average wastewater flow from commercial/ institutional and industrial land use areas is to be estimated as set out in Table 5.1 or by actual documented usage.

TABLE 5.1
EXPECTED VOLUME OF SEWAGE PER DAY*

Place	Estimated Sewage Flow Litres (gallons) Per Day
Assembly Halls	32 (7) per seat
Campsite	80 (18) per campsite
Churches	23 (5) per seat
with kitchen	32 (7) per seat
Construction Camps	225 (50) per person
Day Care Centre	113 (25) per child
Dwellings	675 (150) per bedroom
Golf Clubs	45 (10) per member
with bar and restaurant add	113 (25) per seat
Hospital	
(no resident personnel)	900 (200) per bed
Industrial and Commercial Buildings	
(does not include process water or cafeteria)	45 (10) per employee
(with showers)	90 (20) per employee
Institutions	
(resident)	450 (100) per resident
Laundries	
(coin operated)	1800 (400) per machine
Liquor Licence Establishments	113 (25) per seat
Mobile Home Parks	1350 (300) per space
Motels/Hotels	90 (20) per single bed
Nursing and Rest Homes	450 (100) per resident
Office Buildings	90 (20) per employee
Recreational Vehicle Park	180 (40) per space
Restaurants	
24-Hour	225 (50) per seat
Not 24-Hour	160 (35) per seat

Place	Estimated Sewage Flow Litres (gallons) Per Day
Schools	
Elementary	70 (15) per student
Junior High	70 (15) per student
High School	90 (20) per student
Boarding	290 (65) per student
Service Stations	
(exclusive of cafe)	560 (125) per fuel outlet
Swimming Pools (Public)	
based on design bathing load	23 (5) per person

* Reproduced from the Alberta Private Sewage Treatment and Disposal Regulations, Table 8.5.B.

2. Average Flow Generation Estimates for Planning

For system planning purposes, when specific land uses and zoning are unknown and the requirements of 5.1.1.2 (1) cannot be defined, the recommended lower limits for estimation of average flow generation (to be used for preliminary planning unless the use of other values is justified with more specific or reliable information) are as follows:

(i) Commercial and Institutional Land Uses

The lower limit for Average Flow Generation should be 40 m^3/day/ha (0.46 L/s/ha).

(ii) Industrial Land Uses

The lower limit for average flow generation should be 30 m^3/day/ha (0.35 L/s/ha).

3. Determination of Peak Dry Weather Flow Rate

Peak dry weather flow rates for specific design areas are to be determined by application of a peaking factor (Pf), related to the average flow rate (Q_{AVG} in L/s) in accordance with the following expression to a maximum value of 5.0:

$$Pf = 6.659 (Q_{AVG}{}^{-0.168})$$

Following from this, the peak dry weather flow rate (Q_{PDW} in L/s) may be determined as follows:

$$\begin{aligned} Q_{PDW} &= Pf.Q_{AVG} \\ &= 6.659 (Q_{AVG}{}^{0.832}) \end{aligned}$$

4. Special Considerations - High-Water-Consumption Land Uses

The foregoing guidelines may not be applied to high water consumption land uses such as heavy industry, meat packing plants, breweries, etc. Detailed analysis of the design requirements specific to each development proposal is required in such cases.

5. Residential Components of Commercial Developments

Where proposed commercial developments include discretionary residential components, the sanitary flow generation from the residential component should be determined in accordance with Section 5.1.1.1, and is to be included in the determination of the total generation for the development.

5.1.1.3 Extraneous Flow Allowance - All Land Uses

In computing the total peak flow rates for design of sanitary sewers, the designer should include allowances as specified below to account for flow from extraneous sources.

1. General Inflow/Infiltration Allowance

A general allowance of 0.28 L/s/ha should be applied, irrespective of land use classification, to account for wet-weather inflow to manholes not located in street sags and for infiltration flow into pipes and manholes.

In addition, a separate allowance for inflow to manholes located in street sags should be added as per the next section.

2. Inflow Allowance - Manholes in Sag Locations

When sanitary sewer manholes are located within roadway sags or other low areas, and are thus subject to inundation during major rainfall events, the sanitary design peak flow rate should be increased by 0.4 L/s for each such manhole, which is applicable for manholes which have been waterproofed. For new construction, all sanitary manholes in sag locations are to be waterproofed.

For planning purposes and downstream system design, where specific requirements for an area are unknown, the designer should make a conservative estimate of the number of such manholes which may be installed in the contributing area based on the nature of the anticipated development, and include an appropriate allowance in the design.

3. Others

In areas where weeping tiles are connected to the sanitary sewer system, an additional amount, based on on-site measurements, should be included in the design flow. The designer should also take into account the pipe material and soil type in determining the extraneous flow allowance.

5.1.1.4 Total Peak Design Flow Rates

The total peak design flow rates should be the sum of the peak dry weather flow rates as generated by population and land use, and the rate of all extraneous flow allowances, as determined for the design contributing area.

5.1.2 System Capacity

In general, sewer capacities should be designed for the estimated ultimate tributary population, except in considering parts of the systems that can be readily increased in capacity. For example, the wastewater treatment plant should be designed for a minimum period of 10 years with provision for expansion to handle a 20 or 25-year design flow. Outfall structures, which have high base construction costs, should be designed for the entire design horizon which is usually about 20 to 25 years. The decision is best made based on economic analysis and cost return.

5.1.3 Wastewater Collection and Treatment System

Wastewater collection system including the pumping stations should be designed for peak wastewater design flows.

Aerated lagoon systems should be designed for maximum monthly average daily design flows, with sufficient aeration to maintain a uniform solids concentration in the complete mix cell.

Mechanical wastewater treatment plants should be hydraulically capable of handling the anticipated peak wastewater design flow rates without overtopping channels and/or tankage. From a process point-of-view, however, the design of various components of the plant should be based on the following:

Screening/Grit Removal - Peak wastewater design flow rate.

Primary Sedimentation - Average design flow rates or peak wastewater design flow rate.

Aeration - Maximum monthly average CBOD loading rate in the design year is usually sufficient with predominantly domestic wastes, but the presence of significant industrial waste loadings may create sufficient diurnal variations to warrant consideration. Seasonal variations in domestic and/or industrial CBOD loading rates should also be taken into consideration. Except for short retention treatment systems such as contact stabilization or high rate processes, hydraulic retention time is seldom critical.

Secondary Sedimentation - Peak wastewater design flow rates or peak solids loading rate.

Disinfection System - Peak wastewater design flow rates.

Effluent Filtration - Peak wastewater design flow rates.

5.1.4 Sewer Outfall

Sewer outfall should be designed for peak wastewater design flow rates.

The proper siting and design of the sewer outfall is important in minimizing the impact on receiving water quality. Outfalls should be designed and located so as to obtain the greatest possible dilution of the effluent as quickly as possible during low flow periods.

Dilution is a product of initial mixing of the effluent with surrounding water and subsequent dispersion due to water movement. Initial mixing is enhanced by extending the outfall away from the shore into deeper water and often by incorporating a multiport diffuser to spread the discharge over a larger area and to increase turbulent mixing. Similarly, dispersion is aided by maximizing the separation of the discharged plume from boundary effects of the shoreline or streambed.

5.2 Wastewater Collection

5.2.1 Sewers

5.2.1.1 Materials

The material selected should be adapted to local conditions, such as: character of wastes, possibility of septicity, soil characteristics, exceptionally heavy external loadings, abrasion, hydrogen sulphide corrosion, and similar problems.

Suitable couplings shall be used for joining dissimilar materials.

All sewers should be designed to prevent damage from superimposed live, dead, and frost induced loads. Proper allowance for loads on the sewer should be made because of soil and potential groundwater conditions, as well as the width and depth of trench. Where necessary, special bedding, haunching and initial backfill, concrete cradle, or other special construction should be used to withstand anticipated potential superimposed loading or loss of trench wall stability.

For application in which the wastewater is conveyed under pressure, or in special cases involving excessive surcharge such as inverted siphons, pressure pipes should be used. Pipe and joints should be equal to watermain strength materials suitable for design conditions.

5.2.1.2 Sizing of Sewers

It is normal practice to design sanitary sewers to have a hydraulic capacity such that the sewer is flowing at no more than 80% of the depth when conveying the estimated design peak flow. This is because the maximum velocity is achieved when the flow is at about 0.8 of depth (note: maximum flow occurs when the pipe is flowing at about 0.93 of depth. The reason for this is that as a section approaches full flow, the additional friction resistance caused by the crown of the pipe has a greater effect than the added cross sectional area).

Flow rate at a depth of 80% of the sewer diameter is approximately 86% of the sewer full capacity. Therefore, the required flow capacity for sizing of the sewer is computed using the following relationship:

$$Required\ sewer\ capacity = \frac{Estimated\ design\ flow}{0.86}$$

Manning equation is generally used in sizing the sewers:

$$Q = \frac{1.00}{n} AR^{2/3} S^{1/2}$$

where: Q = Quantity of flow (m^3/s)

n = Roughness coefficient (common value used is 0.013; lower value may be used for PVC pipes based on manufacture's recommendation)

A = Cross sectional area of flow (m^2)

R = Hydraulic Radius (m)

S = Slope (m/m).

5.2.1.3 Changes in Pipe Size

When a smaller sewer joins a large one, the invert of the larger sewer should be lowered sufficiently to maintain the energy gradient. An approximate method for securing these results is to place the 0.8 depth point of both sewers at the same elevation.

5.2.1.4 Location

At the discretion of the municipality, the sewers may be located on the sides of the undeveloped road allowances or on the verges of developed roads.

5.2.1.5 Pressure Testing

Testing of sewers is recommended when high water table is expected or encountered.

The infiltration/exfiltration rate for PVC sewer pipes and fittings may not exceed 4.6 litres per mm diameter of pipe per km length per day. Low-pressure air testing may be permitted to verify this joint tightness when tested to a maximum rate of air loss of 0.0015 ft^3 per minute per ft^2 of internal surface. Test methods to the requirements of Uni-Bell Standard UNI-B-6-90; see Appendix E for test time calculation.

5.2.2 Manholes

Manholes should be durable structures for the purpose of providing convenient access to sewers for observations, inspections, flow monitoring and maintenance operations, at the same time causing a minimum of interference in the hydraulics of the sewer system.

Where corrosion is expected because of either sulphate attack or sulphide, consideration shall be given to the provision of corrosion resistant material or effective protective linings.

5.2.2.1 Location

Manholes should be installed:

- at the end of each line;
- at all changes in grade, size, or alignment;
- at all intersections; and at distances not greater than 120 m for sewers 375 mm or less; or
- 150 m for sewers 450 mm to 750 mm.

However, the limits may be exceeded if suitable modern cleaning equipment is available to handle the larger spacing.

Greater spacing may be allowed in sewers larger than 750 mm.

Cleanouts may be used only for special conditions and may not be substituted for manholes nor installed at the end of laterals greater than 50 m in length.

Manholes should not be located in areas subject to ponding during rainstorms and snowmelt.

5.2.2.2 Sizing

For sewers up to 1050 mm in size, manholes should be constructed with a diameter of at least 1200 mm. For sewers larger than 1050 mm, special type manholes or tee riser manholes may be used. Safety and entry requirements should also be considered when sizing manholes.

5.2.2.3 Drop Manholes

Drop manholes should be used when invert levels of inlet and outlet sewers differ by 600 mm or more. Where the difference in elevation is less than 600 mm, the 0.8 depth point of both sewers should be matched.

Drop manholes should be constructed with an outside drop connection. Inside drop connections (when necessary) shall be secured to the interior wall of the manhole and provide access for cleaning. Drop should be designed to minimize the release of sewer gases.

Due to the unequal earth pressures that would result from the backfilling operation in the vicinity of the manhole, the entire outside drop connection shall be encased in concrete.

5.2.2.4 Channelling and Benching

Good design practice should prevent the depth of flow from being above the sidewalls of the manhole channelling at all times. Therefore, manhole channels should be a smooth continuation of the incoming pipe, the channel height being one-half the pipe diameter for small sewers or three-fourths the pipe diameter for large sewers (375 mm or larger).

Manhole benching should ensure both good footing for workmen and adequate space for minor tools and equipment. Benching should have enough slope for drainage, however to provide safe footing the slope should not exceed 80 mm/m.

No lateral sewer, service connection, or drop manhole pipe should discharge onto the surface of the benching.

5.2.2.5 Frame and Cover

Manhole covers should be designed having the following:

1. Adequate strength to support superimposed loads. Frames and covers are usually cast iron, however lighter weight materials may be used where there is no danger of subjection to heavy loads;

2. Adequate size to facilitate access of equipment and people;

3. A good fit between cover and frame to prevent rattling in traffic;

4. Water tightness between cover and frame to reduce infiltration;

5. Provision for ease of opening (usually a pick notch to pry the cover loose) and an additional pick hole near the edge of the cover;

6. Provision of vent holes; and

7. Resistance to unauthorized entry. The principle defence against a manhole cover being lifted by children is its weight, however during infrequent storm events it is possible that

surcharge and lifting of the cover can occur. Therefore, provision should be made in the design to eliminate the possibility of a person falling into the manhole if the cover has been dislodged.

5.2.2.6 Steps

Manhole steps should be either aluminum or galvanized steel, being wide enough to place both feet on one step. Spacing of steps should be 300 to 400 mm.

To reduce the possibility of feet slipping on manhole steps, the safety-drop type of steps are recommended. For those manholes located within a roadway, and where possible, steps should be aligned so that the person exiting from the manhole should do so facing towards oncoming traffic.

5.2.3 Inverted Siphons

Inverted siphons should have not less than two barrels, with a minimum pipe size of 100 mm. They should be provided with necessary appurtenances for maintenance, convenient flushing, and cleaning equipment. The inlet and discharge structures should have adequate clearances for cleaning equipment, inspection, and flushing. Design should provide sufficient head and appropriate pipe sizes to secure velocities of at least 1 m/s for average design flows. The inlet and outlet details should be so arranged that the flow is diverted to one barrel, and so that either barrel may be cut out of service for cleaning. The vertical alignment should permit cleaning and maintenance.

5.2.4 Wastewater Pump Station

5.2.4.1 General

Wastewater pump stations in general use fall into four types:

1. wet well/dry well;
2. submersible;
3. suction lift; and
4. screw pump.

Once the need for a pump station has been determined, the designer should select the type and location that offers a proper balance between the technical needs, economics, and the environment.

Special consideration should be given to the location of the structure relative to neighbouring development in order to minimize the possible effects of noise and odour.

All weather vehicular access should be provided to all pump stations. Security fencing and access hatches with locks should also be provided.

5.2.4.2 Wet Well/Dry Well Pump Station

1. Structures

Safety ventilation, well separation, access and safety requirements shall be in accordance with the details outlined in Section 4.1.2.

(i) Equipment Removal

Provision should be made to facilitate removing pumps, monitors, and other mechanical and electrical equipment.

(ii) Buoyancy

Where high groundwater conditions are expected, buoyancy of the wastewater pumping station structures should be considered and, if necessary, adequate provisions should be made for protection.

2. Pumps

(i) Protection Against Clogging

Pumps handling wastewater from 750 mm or larger diameter sewers should be preceded by readily accessible bar racks to protect the pumps from clogging or damage. Bar racks should have clear openings as provided in Section 5.3.1.4. Where a bar rack is provided, a mechanical hoist should also be provided.

(ii) Pump Openings

Pumps handling raw wastewater should be capable of passing particles of at least 75 mm in diameter. Pump suction and discharge openings should be at least 100 mm in diameter.

(iii) Priming

The pump should be so placed that under normal operating conditions it will operate under a positive suction head, except as specified in Section 5.2.4.3.

(iv) Electrical Equipment

Electrical systems and components (e.g. motors, lights, cables, conduits, switchboxes, control circuits, etc.) in raw wastewater wet wells, or in enclosed or partially enclosed spaces where hazardous concentrations of flammable gases or vapours may be present, should comply with the Canadian Electrical Code requirements for Class I Group D, Division 1 locations. In addition, equipment located in the wet well should be suitable for use under corrosive conditions. Each flexible cable should be provided with a watertight seal and separate strain relief. A fused disconnect switch located above ground should be provided for the main power feed for all pumping stations. When such equipment is exposed to weather, it should be weather proofed. A 110-volt power receptacle to facilitate maintenance should be provided inside the control panel for lift stations that have control panels outdoors. Ground fault interruption protection should be provided for all outdoor outlets.

(v) Intake

Each pump should have an individual intake. Wet well and intake design should be such as to avoid turbulence near the intake and to prevent vortex formation.

(vi) Dry Well Dewatering

A sump pump equipped with dual check valves should be provided in the dry well to remove leakage or drainage with discharge above the maximum high water level of the wet well. All floor and walkway surfaces should have an adequate slope to a point of drainage. Pump seal leakage shall be piped or channelled directly to the sump. The sump pump should be sized to remove the maximum pump seal water discharge which would occur in the event of a pump seal failure.

(vii) Pumping Rates

The pumps and controls of main pumping stations should be selected to operate at varying delivery rates. Insofar as is practicable, such stations should be designed to deliver as uniform a flow as practicable in order to minimize hydraulic surges. The design flow should be adequate to maintain a minimum velocity of 0.6 m/s in the forcemain.

3. Valves

(i) Suction Line

Suitable shut-off valves should be placed on the suction line of dry pit pumps.

(ii) Discharge Line

Suitable shut-off and check valves should be placed on the discharge line of each pump (except on screw pumps). The check valve should be located between the shut-off valve and the pump. Check valves should be suitable for the material being handled and shall be placed on the horizontal portion of discharge piping except for ball checks, which may be placed in the vertical run. Valves should be capable of withstanding normal pressure and water hammer.

All shut-off and check valves should be operable from the floor level and accessible for maintenance. Outside levers are recommended on swing check valves.

4. Wet Wells

(i) Divided Wells

Where continuity of pumping station operation is critical, consideration should be given to dividing the wet well into two sections, properly interconnected, to facilitate repairs and cleaning.

(ii) Size

The design fill time and minimum pump cycle time should be considered in sizing the wet well. The effective volume of the wet well should be based on design average flow and a filling time not to exceed 30 minutes unless the facility is designed to provide flow equalization/storage. The pump manufacturer's duty cycle recommendations may be utilized in selecting the minimum cycle time. When the anticipated initial flow to the pumping station is less than the design average flow, provisions should be made so that the fill time indicated is not exceeded for initial flows. When the wet well is designed for flow equalization as part of a treatment plant, provisions should be made to prevent septicity. The well and the pumps should also be configured to avoid settlement of solids in the wet well.

(iii) Floor Slope

The wet well floor should have a minimum slope of one-to-one to the hopper bottom. The horizontal area of the hopper bottom should be no greater than necessary for proper installation and function of the inlet.

(iv) Air Displacement

Covered wet wells should have provisions for air displacement such as an inverted "J" tube or other means which vents to the outside.

5. Flow Measurement

Suitable devices for measuring wastewater flow should be provided at all pumping stations. Indicating, totalizing and recording flow measurement devices/instruments should be provided at large pumping stations with peak design flow greater than 50 L/s. Elapsed time meters may be used for pump stations with peak design flow less than 50 L/s.

5.2.4.3 Suction-Lift Pump Station

Pump Priming and Lift Requirements

Suction-lift pumps should be of the self-priming type and should meet the applicable requirements of Section 5.2.4.2. Suction-lift pump stations using dynamic suction lifts may exceed the limits outlined in the following sections if the manufacturer certifies pump performance and submits detailed calculations indicating satisfactory performance under the proposed operating conditions. Such detailed calculations should include static suction-lift as measured from "lead pump off" elevation to centre line of pump suction, friction, and other hydraulic losses of the suction piping, vapour pressure of the liquid, altitude correction, required net positive suction head, and a safety factor of at least 1.8 m.

The pump equipment compartment should be above grade or offset and shall be effectively isolated from the wet well to prevent the humid and corrosive sewer atmosphere from entering the equipment compartment.

1. Self-Priming Pumps

Self-priming pumps should be capable of rapid priming and repriming at the "lead pump on" elevation. Such self-priming and repriming should be accomplished automatically under design operating conditions. Suction piping should not exceed 7.6 m in total length. Priming lift at the "lead pump on" elevation should include a safety factor of at least 1.2 m from the maximum allowable priming lift for the specific equipment at design operating conditions. The combined total of dynamic suction-lift at the "pump off" elevation and required net positive suction head at design operating conditions should not exceed 6.7 m.

2. Vacuum-Priming Pumps

Vacuum-priming pump stations should be equipped with dual vacuum pumps capable of automatically and completely removing air from the suction-lift pump. The vacuum pumps should be adequately protected from damage due to wastewater. The combined total of dynamic suction-lift at the "pump-off" elevation and required net positive head at design operating conditions should not exceed 6.7 m.

5.2.4.4 Submersible Pump Stations

Submersible pump stations should meet the applicable requirements under Section 5.2.4.2, except as modified in this Section.

1. Construction

Submersible pumps and motors should be designed specifically for raw wastewater use, including totally submerged operation during a portion of each pumping cycle and should meet the requirements of the Canadian Electrical Code for such units. An effective method to detect shaft seal failure or potential seal failure should be provided.

2. Pump Removal

Submersible pumps should be readily removable and replaceable without dewatering the wet well or disconnecting any piping in the wet well.

3. Electrical

(i) Power Supply and Control

Electrical supply, control, and alarm circuits should be designed to provide strain relief and to allow disconnection from outside the wet well. Terminals and connectors should be protected from corrosion by location outside the wet well or through use of watertight seals. If located outside, weatherproof equipment should be used.

(ii) Controls

The motor control centre should be located outside the wet well, be readily accessible, and be protected by a conduit seal or other appropriate measures meeting the requirements of the Canadian Electrical Code, to prevent the atmosphere of the wet well from gaining access to the control centre. The seal should be so located that the motor may be removed and electrically disconnected without disturbing the seal.

(iii) Power Cord

Pump motor power cords should be designed for flexibility and serviceability of the Canadian Electrical Code standards for flexible cords in wastewater pump stations. Ground fault interruption protection should be used to de-energize the circuit in the event of any failure in the electrical integrity of the cable. Power cord terminal fittings should be corrosion-resistant and constructed in a manner to prevent the entry of moisture into the cable, should be provided with strain relief appurtenances, and should be designed to facilitate field connecting.

4. Valves

Valves required under Section 5.2.4.2 (4) should be located in a separate valve pit. Valve pits may be dewatered to the wet well through a valved drain line. Check valves that are integral to the pump need not be located in a separate valve pit provided that the valve can be removed from the wet well in accordance with Section 5.2.4.4 (2).

5.2.4.5 Alarm Systems

Alarm systems should be provided for pump stations. The alarm should be activated in cases of high and low liquid levels, power failure, sump pump failure, pump failure, unauthorized entry, or any cause of pump station malfunction. Pumping station alarms should be telemetered to a municipal facility that is manned 24 hours a day. If such a facility is not available and a 24-hour holding capacity is not provided, the alarm should be telemetered to municipal offices during normal working hours and to the home of the responsible person(s) in charge of the lift station during off-duty hours. Audio-visual alarm systems with a self-contained power supply may be installed in some cases in lieu of the telemetering system outlined above, depending upon location, station holding capacity and inspection frequency.

5.2.4.6 Emergency Operation

1. Objective

Wastewater pumping stations should be designed and operated in such a way that equipment breakdown may not result in the discharge of raw or partially treated wastewater to any waters and to protect public health by preventing back-up of wastewater and subsequent discharge to basements, streets, and other public and private property.

2. Emergency Pumping Capability

Emergency pumping capability should be included unless on-system overflow prevention is provided by adequate storage capacity. Emergency pumping capability may be accomplished by connection of the station to at least two independent power grids, or by provision of portable or in-place internal combustion engine equipment which will generate electrical or mechanical energy, or by the provision of portable pumping equipment. Such emergency standby systems should have sufficient capacity to start up and maintain the total related running capacity of the station. Regardless of the type of emergency standby system provided, a riser from the forcemain with rapid connection capabilities and appropriate valving should be provided for lift stations to hook up portable pumps.

3. Emergency High Level Overflows

For use during possible periods of extensive power outages, mandatory power reductions, or uncontrollable emergency conditions, consideration should be given to providing a controlled, high-level wet well overflow to supplement alarm systems and emergency power generation in order to prevent backup of wastewater into basements, or other discharges which may cause severe adverse impacts on public interests, including public health and property damage. Where a high level overflow is utilized, consideration should also be given to the installation of storage/detention tanks, or basins, which should be made to drain to the station wet well. Overflows should be considered only in conjunction with emergency pumping capability as outlined in Section 5.2.4.6 (2).

4. Equipment Requirements

(i) General

The following general requirements should apply to all internal combustion engines used to drive auxiliary pumps, service pumps through special drives, or electrical generating equipment:

(a) Engine Protection

The engine must be protected from operating conditions that would result in damage to equipment. Unless continuous manual supervision is planned, protective equipment should be capable of shutting down the engine and activating an alarm on site and as provided in Section 5.2.4.5. Protective equipment should monitor for conditions of low oil pressure and overheating, except that oil pressure monitoring will not be required for engines with splash lubrication.

(b) Size

The engine should have adequate rated power to start and continuously operate under all connected loads.

(c) Fuel Type

Reliability, and ease of starting, especially during cold weather conditions, should be considered in the selection of the type of fuel.

(d) Engine Ventilation

The engine should be located above grade with adequate ventilation of fuel vapours and exhaust gases.

(e) Routine Start-Up

All emergency equipment should be provided with instructions indicating the need for regular starting and running of such units at full loads.

(f) Protection of Equipment

Emergency equipment should be protected from damage at the restoration of regular electrical power.

(ii) Engine-Driven Pumping Equipment

Where permanently installed or portable engine-driven pumps are used, the following requirements in addition to general requirements should apply:

(a) Pumping Capacity

Engine-driven pumps should meet the design pumping requirements unless storage capacity is available for flows in excess of pump capacity. Pumps should be designed for anticipated operating conditions, including suction lift if applicable.

(b) Operation

The engine and pump should be equipped to provide automatic start-up and operation of pumping equipment unless manual start-up and operation is justified. Provisions should also be made for manual start-up. Where manual start-up and operation is justified, storage capacity and alarm system must meet the requirements of Section 5.2.4.6 (4) (ii) (c).

(c) Portable Pumping Equipment

Where part or all the engine-driven pumping equipment is portable, sufficient storage capacity with alarm system should be provided to allow time for detection of pump station failure and transportation and hook-up of the portable equipment.

(iii) Engine-Driven Generating Equipment

Where permanently installed or portable engine-driven generating equipment is used, the following requirements should apply in addition to general requirements:

(a) Generating Capacity

- Generating unit size should be adequate to provide power for pump motor starting current and for lighting, ventilation, and other auxiliary equipment necessary for safety and proper operation of the lift station.
- Special sequencing controls should be provided to start pump motors unless the generating equipment has capacity to start all pumps simultaneously with auxiliary equipment operating.

(b) Operation

Provisions should be made for automatic and manual start-up and load transfer unless only manual start-up and operation is justified. The generator should be protected from operating conditions that would result in damage to equipment. Provisions should be considered to allow the engine to start and stabilize at operating speed before assuming the load. Where manual start-up and transfer is justified, storage capacity and alarm system should meet the requirements of Section 5.2.4.6 (4) (iii) (c).

(c) **Portable Generating Equipment**

Where portable generating equipment or manual transfer is provided, sufficient storage capacity with alarm system should be provided to allow time for detection of pump station failure and transportation and connection of generating equipment. The use of special electrical connections and double throw switches are recommended for connecting portable generating equipment.

5.2.5 Forcemains

5.2.5.1 Velocity and Diameter

At design pumping rates, a cleansing velocity of at least 0.6 m/s should be maintained.

5.2.5.2 Air and Vacuum Relief Valve

An air relief valve should be placed at high points in the forcemain to prevent air locking. Vacuum relief valves may be necessary to relieve negative pressures on forcemains. The forcemain configuration and head conditions should be evaluated as to the need for and placement of vacuum relief valves.

5.2.5.3 Termination

Forcemains should enter the gravity sewer system at a point not more than 600 mm above the flow line of the receiving manhole.

5.2.5.4 Design Pressure

The forcemain and station piping should be designed to withstand water hammer pressures and associated cyclic reversal of stresses that are expected with the cycling of wastewater lift stations. Surge protection systems should be evaluated.

5.2.6 Security of Open Trenches and Excavations

In order to ensure public safety, Local Authorities responsible for the construction should secure open trenches and excavations during non-working periods by installing fences/barricades and/or warning lights/signs.

5.3 Wastewater Treatment

5.3.1 Mechanical Wastewater Treatment

5.3.1.1 Site Selection

1. Plant Location

Some of the factors which should be taken into consideration when selecting a new plant site are as follows:

(i) setback distances from land use surrounding plant site [see Section 4.2.3.2 (1)];

(ii) susceptibility of site to flooding [See Section 4.2.1.3 (3)];

(iii) prevailing wind direction; and

(iv) adequacy of site for future expansion.

2. Plant Layout

Plant buildings should be situated to provide adequate allowances for future expansions of the various treatment sections. The plant should also be oriented so that the best advantage can be taken of the prevailing wind and weather conditions to minimize odour, noise, misting, freezing problems, energy consumption, and other environmental impacts. The plant layout should also allow for the probability of snow drifting. Entrances, roadways and open tankage should be located so that the effect of snow drifting on operations will be minimized.

Processing units should be arranged in a logical progression to avoid the necessity for major pipelines or conduits to transmit wastewater, sludges, or chemicals from one module to the next, and also to provide for convenience of operation and ease of flow splitting for proposed and future treatment units.

Vehicular access should be sufficient to allow for the largest anticipated delivery or disposal, with allowance made to accommodate vehicle turning and forward exit from the plant site.

3. Provision for Expansion

In addition to the general site considerations, there are a number of allowances needed to provide for economical and practical expansion of the wastewater treatment facilities. Key provisions include:

(i) Design of on-site pumping stations such that their capacity can be increased and/or parallel facilities constructed without the need for major disruption of the plant's operation;

(ii) Layout and sizing of channels and plant piping such that additional treatment units can be added or increases in loading rates accommodated. Similarly, the layout of buildings and tankage should accommodate the location of the future stages of expansion;

(iii) Space provision within buildings to provide for replacement of equipment with larger capacity units. This is particularly important with equipment such as pumps, blowers, boilers, heat exchanges, etc. Adequate working space should be provided around equipment, and provision made for the removal of equipment; and

(iv) Sizing of inlet and outlet sewers to account for the ultimate plant capacity. Provided that problems will not occur with excessive sedimentation in the sewers, these sewers should be sized for the ultimate condition. With diffused outfalls, satisfactory port velocities can often be obtained by blocking off ports which will not be required until subsequent expansion stages.

5.3.1.2 Plant Hydraulics

1. Wastewater Pumpage

Raw wastewater and any intermediate wastewater pump stations associated with wastewater treatment works should be capable of conveying the peak wastewater flow rates to downstream treatment units. Pumping equipment should also be designed so that downstream treatment units are not subjected to unnecessary surging. This is best achieved by providing variable capacity, or multiple fixed capacity pumps, so that pump discharge rates will closely match the sewage inflow rate. [See also Section 5.2.4.2 (2) (vii)].

2. Channel Flow

Channels should be designed to convey the initial and ultimate range of flows expected. To avoid solids build-up, the following scouring velocities should be developed in normally used channels at least once per day:

Wastewater containing grit - 0.9 m/s
Wastewater containing floc suspensions - 0.45 to 0.60 m/s

Where the above scouring velocities cannot be obtained, channels may be aerated to prevent solids deposition.

3. Flow Division

Within wastewater treatment plants, there will invariably be situations where flow splitting is necessary. Unless certain precautions are taken, the flow will not split in the proportions desired over the full flow range, or the flow may split properly, but the organic load will not be divided in the same proportion.

To ensure that the organic load splits in the same proportion as the flows, the suspended solids should be homogeneously dispersed throughout the liquid and the relative momentum of all particles should be approximately equal at the point of diversion. Some turbulence is therefore desirable before each point of diversion. The following methods can be used to produce homogeneity:

- mechanical mixers;
- diffused aeration;
- bottom entrance into splitting box; or
- bar racks or posts in channels.

4. Plant Hydraulic Gradient

The hydraulic gradient of all gravity flow and pumped waste streams within the wastewater treatment plant, including bypass channels, should be prepared to ensure that adequate provision has been made for all head losses. In calculating the hydraulic gradient, changes in head caused by all factors should be considered, including the following:

(i) head losses due to channel and pipe wall friction;

(ii) head losses due to sudden enlargement or sudden contraction in flow cross section;

(iii) head losses due to sudden changes in direction, such as at bends, elbows, Y-branches and tees;

(iv) head losses due to sudden changes in slope, or drops;

(v) head losses due to obstructions in conduit;

(vi) head required to allow flow over weirs, through flumes, orifices and other measuring, controlling, or flow division devices;

(vii) head losses caused by flow through comminutors, bar screens, tankage, filters and other treatment units;

(viii) head losses caused by air entrainment or air binding;

(ix) head losses incurred due to flow splitting along the side of a channel;

(x) head increases caused by pumping; and

(xi) head allowances for expansion requirements and/or process changes.

Consequences of excessive or inadequate allowances for head losses through wastewater treatment works should be noted. If pumpage is required, excessive head loss allowances result in energy wastage. If inadequate head loss allowances are made, operation will be difficult and plant expansion more costly.

5.3.1.3 Wastewater Characterization

Detailed wastewater characterization studies should be undertaken whenever existing data is limited or of suspect quality. Industrial discharges to municipal systems can significantly alter the characteristics and treatability of domestic wastewater. Wastewater containing industrial discharges should therefore be thoroughly characterized before selecting and designing biological treatment process units. Ideally, the undiluted industrial wastewater itself should be characterized so that any spikes originating from that source can be accounted for in the design of the process units. Oil and grease, pH, phosphorus and nitrogen levels should also be determined prior to selection and design of the treatment process.

Where it is found that sewage strengths vary significantly over the year due to excessive infiltration/inflow, population variations and/or seasonal changes in industrial or commercial operations, estimates should be made of the expected average, maximum, and minimum BOD and suspended solids concentrations in the sewage for each month of the year. If nitrification is required, short-and-long-term variations in ammonia and total Kjeldahl nitrogen concentrations should also be estimated.

Biological treatment process units are generally designed using total BOD loadings, however, in some cases soluble BOD loadings may be used where recommended by equipment suppliers. In such cases it is generally assumed that soluble BOD represents a certain fraction of the total BOD. Because wastewater characteristics can vary significantly, the actual ratio of soluble to total BOD should be determined whenever soluble BOD is used for design purposes.

Optimum growth of the microorganisms is dependent on the supply of essential nutrients and trace elements. In addition to carbon, the two most critical elements are nitrogen and phosphorus. To encourage the growth of the organism, it is advisable to maintain a BOD:N:P ratio of 100:5:1. Failure to maintain a balanced nutrient level could result in operational problem. If necessary, nutrients may have to be added to the wastewater to provide a balanced level for microbial growth.

While the design of the plant to treat the domestic component of the total BOD may be straightforward, the potential difficulties with biological stabilization of the industrial wastewater flows should be recognized. Therefore, the industrial flow component should be characterized by using the following ratio:

$$\frac{\text{BOD of the industrial flow at 5 days and } 20^0\text{C}}{\text{BOD of the industrial flow at 20 days and } 20^0\text{C}}$$

Many industrial wastes also contain substances that may exert toxic effects on the organisms. Phenol, cyanide, ammonia, sulphide, heavy metals and many organic compounds may completely inhibit the microbial activity if these concentrations exceed the limit which an be tolerated by the micro-organisms. Thus, waste characterization is extremely important in the design of the plant.

5.3.1.4 Preliminary Treatment

Preliminary treatment consists of screening and grit removal.

Screening is provided as the first treatment stage for the protection of plant equipment against blockage, reduced operating efficiency, or physical damage.

Grit removal is required to prevent the undue wear of machinery and unwanted accumulation of solids in channels, settling tanks and digesters.

1. Screening Devices

(i) Coarse Screens

(a) Where Required

Protection for pumps and other equipment should be provided by trash racks, coarse bar racks, or coarse screens.

(b) Design and Installation

- *Bar Spacing*

  Clear openings between bars should be no less than 25 mm for manually cleaned screens. Clear openings for mechanically cleaned screens may be greater than 6 mm. Maximum clear openings should be 50 mm.

- *Slope and Velocity*

  Manually cleaned screens should be placed on a slope of 30 to 45 degrees from the horizontal.

  Approach velocities should be no less than 0.5 m/s to prevent settling; and no greater than 1 m/s to prevent forcing material through the openings, during normal variations inflow conditions.

- *Channels*

  Dual channels should be provided and equipped with the necessary gates to isolate flow from any screening unit. Provisions should also be made to facilitate dewatering each unit. The channel preceding and following the screen should be shaped to eliminate standing and settling of solids.

- *Auxiliary Screens*

  Where a single mechanically cleaned screen is used, an auxiliary manually cleaned screen should be provided. Where two or more mechanically cleaned screens are used, the design should provide for taking any unit out of service without sacrificing the capability to handle the peak design flows.

- *Invert*

  The screen channel invert should be 75-150 mm below the invert of the incoming sewer.

- *Flow Distribution*

  Entrance channels should be designed to provide equal and uniform distribution of flow to the screens.

- *Backwater Effect on Flow Metering*

  Flow measurement devices should be selected for reliability and accuracy. The effect of changes in backwater elevation, due to intermittent cleaning of screens, should be considered in locations of flow measurement equipment.

- *Freeze Protection*

  Screening devices and screening storage areas should be protected from freezing.

- *Screenings Removal and Disposal*

  A convenient and adequate means for removing screenings should be provided. Hoisting or lifting equipment may be necessary depending on the depth of pit and amount of screenings or equipment to be lifted.

  Facilities should be provided for handling, storage, and disposal of screenings. Screenings should be disposed of at the sanitary landfill.

  Manually cleaned screening facilities should include an accessible platform from which the operator may rake screenings easily and safely. Suitable drainage facilities should be provided for both the platform and the storage area.

(ii) Fine Screens

(a) General

Fine screens should have openings of approximately 1.5 mm. The amount of material removed by fine screens is dependent on the waste stream being treated and screen opening size.

Fine screens should not be considered equivalent to primary sedimentation but may be used in lieu of primary sedimentation where subsequent treatment units are designed on the basis of anticipated screen performance. Selection of screen capacity should consider flow restriction due to retained solids, frequency of cleaning, and extent of cleaning. Where fine screens are used, additional provision for removal of floatable oils and greases should be considered.

(b) Design

A minimum of two fine screens should be provided, each unit being capable of independent operation. Capacity should be provided to treat peak design flow with one unit out of service.

Fine screens should be preceded by a coarse bar screening device. Fine screens should be protected from freezing and located to facilitate maintenance.

2. Grit Removal Facilities

Grit removal is usually accomplished by grit channels or aerated grit chambers. Vortex-type (paddle or jet induced vortex) is another not so common type of device used for grit removal.

(i) Grit Channels

(a) Where Required

In advance of pumping or treating units behind screening devices.

(b) Design and Installation

- *Number of Channels*

  At least two (with one out-of-service, there should be enough capacity in the remaining unit to handle the peak design flow). Provision should be made for isolating and dewatering each unit.

- *Velocity*

  Channels should be designed to control velocities during normal variations in flow as close as possible to 0.3 m/s.

- *Channel Length and Width*

  The length should be adequate to settle 0.2 mm particle with a specific gravity of 2.65 plus 50% allowance for inlet and outlet turbulence. Channel width should be greater than 375 mm.

- *Grit Storage*

  With permanently positioned weirs, the weir crest should be kept 150 to 300 mm above the grit channel invert to provide for storage of settled grit (adjustable weir plates are recommended as they can be moved to prevent the sedimentation of organic solids following grit cleaning).

(ii) Aerated Grit Chambers

(a) Design and Installation

Aerated grit chambers for the removal of 0.2 mm, or larger, particles with specific gravity of 2.65, may be designed in accordance with the following parameters:

- *Detention Time*

  2 to 5 minutes at peak design flow rate (the longer retention times provide additional benefit in the form of pre-aeration).

- *Air Supply*

  4.5 to 12 L/m.s, via wide band diffusion header positioned lengthwise along one wall of tank.

- *Tank Dimensions*

  Lower limit of above aeration rates generally suitable for chambers up to 3.7 m deep and 4.3 m wide; wider, or deeper chambers require aeration rates in the upper end of the above range; long, narrow aerated grit chambers are generally more efficient than short chambers and produce cleaner grit; length/width ratio normally is 1.5:1 to 2:1, but up to 5:1 may be used; depth/width ratio 1:1.5 to 1:2.

- *Desired Velocities*

  Surface velocity should be 0.45 to 0.6 m/s.

- *Grit Handling*

  Grit chambers should be provided with mechanical equipment for hoisting or transporting grit to ground level. Impervious, non-slip, working surfaces with adequate drainage should be provided for grit handling areas. Grit transporting facilities should be provided with protection against freezing and loss of material.

- *Grit Washing*

  Depending upon the method of removal and ultimate disposal, the grit may have to be washed after removal by devices of the type discussed in the previous section.

- *Multiple Units*

  Generally not required unless economically justifiable, or where grit removal method requires bypassing of chamber.

5.3.1.5 Primary Treatment

Primary treatment consists of pre-aeration settling (sedimentation) to remove readily settleable solids, floating materials and scum from raw sewage. This is an important process in sewage treatment, as it reduces the suspended solids content and the load on the biological treatment units.

Sedimentation may be accomplished in horizontal or vertical flow tanks. In a horizontal flow tank, the sewage enters at one end and leaves at the other end. In a vertical flow tank, sewage enters at the centre and flows to periphery of the tank. Sludge settles to the tank floor and is removed mechanically into hoppers where subsequent withdrawal occurs.

Accumulation of scum is to be expected in the primary tank; and a scum baffle/skimmer bar is necessary to prevent the scum from discharging with the effluent.

The tank sizing should reflect the degree of solids removal needed and the need to avoid septic conditions during low flow periods. Sizing of the clarifier should be based on both the average design and peak design flow conditions, and the larger area determined should be used.

Maintenance provisions, including access to equipment, lighting, hose, bibs, etc. should also be provided.

- *Design Criteria*

  - (i) Minimum water depth - 2.1 m
  - (ii) Depth to length ratios (rectangular tank) - 1/10 to 1/30
  - (iii) Surface loading for tanks not receiving return sludge; based on average design flow - < 0.47 $L/s/m^2$
  - (iv) Surface loading for tanks not receiving return sludge; based on peak design flow - 0.71 to 1.42 $L/s/m^2$
  - (v) Surface loading for tanks receiving return sludge; based on peak design flow - < 0.47 $L/s/m^2$
  - (vi) Weir overflow rate - 1.74 to 5.21 L/m/s
  - (vii) For scum removal, a scum baffle extending at least 150 mm below the surface is necessary close to the overflow weir.

5.3.1.6 Secondary Treatment

1. General

The objective of secondary treatment (mechanical) is to achieve the effluent standards as specified in section 3.1.2, Table 3.1. It may be accomplished in a suspended growth system, a fixed film system or a coupled system. The following sections will discuss the use of two suspended growth systems: continuous-flow activated sludge process, sequencing batch reactors and one fixed film process, rotating biological contactors.

An integral component of the secondary treatment is the secondary clarifier, which will be discussed in Section 5.3.1.8.

2. Suspended Growth Systems

(i) Continuous-flow activated sludge process

(a) General

The activated sludge process and its various modifications may be used where wastewater is amenable to biological treatment. This process requires close attention and competent operating supervision, including routine laboratory control. These requirements should be considered when proposing this type of treatment.

(b) Pre-treatment

The minimum level of pre-treatment should include grit removal and screening. Primary settling tanks are required unless demonstrated otherwise. If primary tanks are not provided, then the downstream units should be adequately sized.

(c) Types

A number of modifications of the activated sludge process have been developed. The major types of continuous-flow activated sludge processes include: plug flow (conventional), complete mix, extended aeration, contact stabilization and step feed systems.

(d) Sludge Bulking Control

The control of sludge bulking condition should be a main design objective of an activated sludge system. The use of the following process configuration should be considered:

- Plug-flow reactor
- If a complete-mix system is proposed, a selector basin should be constructed upstream of the complete mix basin. Complete-mix activated sludge systems are generally more vulnerable to sludge bulking for treating domestic wastes.

In addition, provision for addition of chlorine or other chemicals to selectively eliminate filamentous bacteria should be included. The chlorine dosage should range from 3 to 15 kg Cl_2 per 1000 kg MLSS/d. The chlorine addition locations should be chosen such that the chlorine is added efficiently and the biomass receives adequate chlorine exposure.

(e) Winter Protection

Due design considerations should be given to minimizing heat loss and to protecting against freezing during winter. Possible design approaches to reduce heat loss during winter conditions include use of diffused aeration instead of surface aeration, reduced surface area by using a deeper tank, tank insulation and provision of covers. If severe climatic conditions are expected, heat loss calculations should be included in the design to assess the aeration basin temperatures under low ambient and wastewater temperatures and low organic loading conditions.

(f) Aeration Basins

- *Sizing*

  The size of the aeration basins should be determined based on the design sludge age using the maximum monthly average BOD loading in the design year. Table 7.2 shows the generally accepted range of sludge age and other parameters such as F/M ratio, mixed liquor suspended solids, aeration tank detention time, aerator loading and sludge recycle ratio for design of the various modifications of the activated sludge process. Consideration should be given to the low loading conditions in the initial operating period.

- *Number of Units*

  Multiple tanks capable of independent operation should be provided for all plants.

- *Dimensions*

  The dimensions of each independent mixed-aeration tank or return sludge re-aeration tank should be chosen so as to maintain effective mixing and utilization of air when diffused air is used. Liquid depths should not be less than 3 m, except in special design cases.

- *Controls*

  Inlets and outlets for each aeration tank unit should be suitably equipped with valves, gates, weirs or other devices to permit controlling the flow to any unit and to maintain reasonably constant liquid levels. The hydraulic capacity of the system shall permit the maximum instantaneous hydraulic load to be carried with any single aeration tank unit out of operation.

- *Conduits*

  Channels and pipes carrying liquids with solids in suspension should be designed to maintain self-cleaning velocities or should be agitated to keep such solids in suspension at all rates of flow within the design limits.

- *Measuring Devices*

  Devices should be installed for measuring and indicating flow rates of influent sewage, return sludge, sludge wasting, dissolved oxygen, and air to each aeration tank.

TABLE 5.2
TYPICAL ACTIVATED SLUDGE DESIGN PARAMETERS

Process Modification	Flow Regime	Food to Micro-organism Ratio $\frac{\text{(g BOD/d)}}{\text{(g MLSS)}}$[d]	Sludge Age (days)	Mixed-Liquor Suspended Solids (mg/l)	Detention Time (hr)	Organic Loading (g BOD/d m^3 tank volume)	Activated Sludge Return Ratio[c]
Conventional	Plug	0.2 to 0.4	5 to 15	1500 to 3000	4 to 8	350 to 650	0.25 to 0.5
Complete mix	Complete mix	0.2 to 0.6	5 to 15	2000 to 5000	3 to 5	350 to 1900	0.25 to 1.0
Step aeration	Plug	0.2 to 0.6	5 to 15	2000 to 3500	3 to 5	350 to 1000	0.25 to 0.75
Contact stabilization	Plug or complete mix	0.2 to 0.6	5 to 15	1000 to 4000[a] 4000 to 10 000[b]	0.5 to 1.5[a] 3 to 6[b]	500 to 1200	0.25 to 1.0
Extended aeration	Plug or complete mix	0.05 to 0.15	10 to 30	2000 to 6000	10 to 24	150 to 400	0.75 to 1.0
High rate	Complete mix	0.4 to 1.5	5 to 10	6000 to 8000	1 to 3	1600 to 4000	0.25 to 0.5
High-purity oxygen systems	Complete mix reactors in series	0.2 to 1.2	3 to 10	3000 to 6000	1 to 5	1600 to 4000	0.25 to 0.5

Notes:

a	Contact tank
b	Stabilization tank
c	1.5 to 2.0 is required for nitrifying facilities
d	< 0.2 is required for nitrifying facilities

- *Conduits*

  Channels and pipes carrying liquids with solids in suspension should be designed to maintain self-cleaning velocities or should be agitated to keep such solids in suspension at all rates of flow within the design limits.

- *Measuring Devices*

  Devices should be installed for measuring and indicating flow rates of influent sewage, return sludge, sludge wasting, dissolved oxygen, and air to each aeration tank.

- *Freeboard*

  Aeration tanks should have a freeboard of at least 0.6 m. Greater heights are desirable. Aeration tanks with mechanical aerators require a minimum freeboard of 1 m.

- *Foam Control*

  Foam control devices should be provided for aeration tanks. Suitable spray systems or other appropriate means will be acceptable. If potable water is used, adequate backflow prevention should be provided on the water lines. The spray lines should have provisions to prevent damage by freezing, where appropriate.

- *Drain and Bypass*

  Provision should be made for dewatering each aeration tank for cleaning and maintenance. The dewatering system should be sized to permit removal of the tank contents within 24 hours. If a drain is used, it should be provided with a control valve. The dewatering discharge should be upstream of the activated sludge process. Provision should be made to isolate each aeration tank without disrupting flow to other aeration tanks.

(g) Activated Sludge Return Equipment

The minimum return sludge rate of withdrawal from the secondary clarifier is a function of the concentration of suspended solids in the aeration tank, the settleability of these solids, and the length of time these solids are retained in the secondary clarifier. The rate of sludge return expressed as a ratio of the average design flow should be variable within the limits set forth in Table 5.2. Separate sludge return lines and pumps should be provided for each clarifier; the system should also be equipped with mechanical or electrical variable speed drive to vary the output of the pump. The pump should be designed for 50 to 125% return of sludge.

Provision should be made in the return lines for the addition of chlorine to the return sludge for controlling sludge bulking.

(h) Waste Activated Sludge Equipment

In designing waste activated control facilities, flexibility should be provided so that the excess activated sludge may be wasted from the return activated sludge lines or directly from the aeration tank. While wasting from the return lines gives a more concentrated sludge, wasting directly from the mixed liquor provides a simpler process control.

The waste activated sludge pumps and pipelines should be sized based on the expected maximum sludge production rates and minimum sludge concentrations. For installations where sludge wastage is not continuous, the sizes of the pumps and pipelines should be increased to handle the sludge wastage during the expected wasting period.

(i) Safety

Handrails should be provided around all aeration tanks and clarifiers and conform to the safety provision of the Alberta Occupational Health and Safety Act and Regulations.

The following safety equipment should be provided near aeration tanks and clarifiers:

- safety vests
- lifelines and rings
- safety poles.

Walkways near aeration tanks should have a roughened surface or grating to provide safe footing.

Sufficient lighting should be provided to permit safe working conditions near aeration tanks and clarifiers at night.

(ii) Sequencing Batch Reactors

(a) General

The sequencing batch reactor (SBR) is a fill and draw activated sludge treatment system. It includes a generic system of variable volume activated sludge in which aeration, sedimentation and decant are combined in a single reactor. Consequently, there are no dedicated secondary clarifier or associated return sludge facilities. The SBR technology is suitable for small installations.

(b) Pre-treatment

The minimum level of pre-treatment should include grit removal and screening.

(c) Types

The SBR systems can be classified under two main types: (1) intermittent feed and intermittent discharge (IFID), and (2) continuous feed and intermittent discharge (CFID).

The IFID systems are sometimes referred to the conventional SBR systems. The common characteristics of all IFID systems is that the influent flow to the reactor is discontinued for some portion of each cycle. The IFID reactor treats the influent wastewater through a succession of operating steps, namely fill, react, settle, draw and idle. The liquid volume inside the SBR increases from a set minimum volume to a predetermined maximum volume during the fill period. Mixing and/or aeration may be provided during this fill step. During the react period, flow to the tank is discontinued and aeration and/or mixing are provided, while sufficient time is allowed for the microbial reactions to take place. During the settle period, quiescent conditions are initiated and the biomass is allowed to flocculate and settle prior to removal. During the draw or decant period, the treated and clarified supernatant is removed from the reactor to the minimum volume level. During the idle period, which is normal component in multi-reactor installations, biomass is retained in the reactor but no waste is treated. During this period, excess biomass may be removed from the tank to maintain the desired sludge age.

The CFID reactors receive wastewater during all phases of the treatment cycle. Because it has continuous fill, it has no separate fill and idle periods. The CFID reactors always have a pre-reaction compartment at the influent end terminating in a baffle.

(d) Winter Protection

The winter protection requirements for the SBR systems are higher than the continuous flow activated sludge systems because of the longer total retention times. In addition to the provisions stated for Suspended Growth Systems (subsection 2.i.e), further considerations should be given to the possibility of freezing of equipment and impact of frozen scum on the proposed decanter system.

(e) SBR Basins

- *Sizing*

  The size of the SBR basins should be determined based on the design aerobic sludge age or aerobic food to microorganism (F/M) ratio, using the maximum monthly average BOD loading in the design year. The aerobic sludge age (or F/M ratio) is determined based on the total system sludge age (or F/M) adjusted based on the aerate (aerate fill plus react) in the operating cycle. The aerobic sludge age and F/M ratio should fall within the acceptable range stated in 5.2. The MLSS levels of an SBR change throughout the operating cycle. The selected MLSS levels in calculating sludge age or F/M ratios should correspond to the levels during the react period.

- *Dimensions*

  A key design consideration with CFID systems is minimization of short-circuiting between influent and effluent. The reactor should be rectangular in shape with length to width ratios of at least 2:1. Baffling should also be provided. The length to width ratio is generally less critical for IFID system but the exact dimensions may be affected by the choice of influent distribution system.

- *Liquid Depths*

  The top liquid depths should not be less than 3 m, except in special design cases. In most practical cases, the top liquid depths should range between 4 to 6 m. The bottom liquid depth should be designed based on the required fill volume to handle peak flow conditions. The bottom liquid depths should be decided based on the expected sludge settleability.

- *Number of Units*

  An IFID system must comprise a minimum of two SBR tanks or a storage tank and an SBR tank to accommodate continuous inflow. One CFID reactor is adequate to handle continuous flow. However, multiple tanks capable of independent operation should be provided for all plants.

- *Controls*

  Inlets and outlets for each aeration tank unit should be suitably equipped with valves, gates, weirs or other devices to permit controlling the flow to any unit and to maintain reasonably constant liquid level. The hydraulic capacity of the system shall permit the maximum instantaneous hydraulic load to be carried with any single aeration tank unit out of operation.

- *Measuring Devices*

  Devices should be installed for measuring and indicating flow rates of influent sewage, sludge wasting, dissolved oxygen, and air to each SBR tank.

- *Freeboard*

  SBR tanks should have a freeboard of at least 0.6 m. Greater heights are desirable. Aeration tanks with mechanical aerators require a minimum freeboard of 1 m.

- *Foam Control*

  Foam control devices should be provided for aeration tanks. Suitable spray systems with provision for chlorine addition or other appropriate means will be acceptable. If potable water is used, adequate backflow prevention should be provided on the water lines. The spray lines should have provisions to prevent damage by freezing, where appropriate.

- *Drain and Bypass*

  Provision should be made for dewatering each SBR tank for cleaning and maintenance. The dewatering system should be sized to permit removal of the tank contents within 24 hours. If a drain is used, it should be provided with a control valve. The dewatering discharge should be upstream of the SBR process. Provision should be made to isolate each SBR tank without disrupting flow to other aeration tanks.

- *Overflow*

  An overflow system should be provided in the SBR basins to handle extreme flow conditions or equipment malfunction conditions.

(f) Decanter

There are various decanter designs of varying sophistication and complexity proposed by the SBR equipment suppliers. In exposed installations where severe climatic conditions, winter protection should be a major consideration in selecting the decanter design.

Among the recent types are: (i) floating decanter, (ii) fixed decanter, and (iii) mechanically actuated surface skimmer. If a fixed decanter is proposed, longer duration of the settle period should be allowed to ensure that the sludge blanket is located low enough to start each decant cycle.

The decanter should have positive control to prevent solids entry into the decanter during aerate and settle period. The common solids excluding decanters include those: (i) incorporating a spring loaded solids excluding valve, (ii) physically removed from the mixed liquor except during decant period, and (iii) mechanically closed when not in use by a hydraulic or electric motor. If positive control is not provided, the effluent from the decanter should be recycled for at least the first several minutes before discharge. The decanter should have positive control against the entry of scum during the settle period; the decanter should also have a scum baffle to prevent scum from exiting with the effluent.

The size of the decanter should be determined based on the fill volume and the decant period. The decant period is generally **one** half to one hour.

(g) Waste Activated Sludge Equipment

The wasting of excess activated sludge is generally discontinuous for SBR system. The size of the waste activated sludge equipment should be decided based on the expected wastage period. Consideration should be given to the practical number of operating cycles during the working hours of the operator.

(h) Aeration Equipment

When choosing the aeration equipment for a SBR system, consideration should be given to the intermittent aeration conditions and the possibility of diffuser clogging. Because aeration takes place only during part of the operating cycle, the aeration equipment should be sized such that the required oxygen transfer can be provided during the react/fill and react periods. The temporal variation of oxygen requirements should also be considered.

(i) Downstream Facilities

The design of downstream facilities should allow for the intermittent discharge of SBR effluent. Note that the average decant rate is generally higher than the design peak flow rate because of the intermittent discharge. If the decanting devices are of varying flow rate design, the peak flow rate in the beginning of the decant cycle should be used to determine the hydraulic capacity of the downstream facilities. The impact of the intermittent effluent discharge on the downstream facilities such as UV disinfection should be taken into consideration. An effluent equalization basin should be provided, when appropriate.

(j) Safety

Handrails should be provided around all aeration tanks and clarifiers and conform to the safety provision of the Alberta Occupational Health and Safety Act and Regulations.

The following safety equipment should be provided near aeration tanks and clarifiers:

- safety vests
- lifelines and rings
- safety poles.

Walkways near aeration tanks should have a roughened surface or grating to provide safe footing.

Sufficient lighting should be provided to permit safe working conditions near aeration tanks and clarifiers at night.

3. Fixed Film Systems

(i) Rotating Biological Contactor

(a) General

The rotating biological contactor (RBC) process may be used where wastewater is amenable to biological treatment.

(b) Pre-treatment

Primary clarifiers should be provided ahead of the RBC process to minimize solids settling in the RBC tanks. If the influent contains appreciable amount of sulphide greater than 0.5 mg/L), pre-aeration should be provided upstream of the RBC process.

(c) Media Types

The media used for RBCs are manufactured of high-density polyethylene and are provided in different configurations or corrugated patterns.

The types of media are classified based on the area of media on the shaft and are commonly termed as standard density, medium-density and high-density. Standard density media have surface area of 9300 m^2 per 8.23 m shaft and should normally be used in the lead stages of an RBC process train. Medium and high-density media have surface areas of 11 150 to 16 700 m^2 per shaft and should be used only after the second shaft.

(d) Staging

RBC plants should be designed in multiple stages with sufficient operational flexibility to split incoming flows between stages during peak loading periods so as not to exceed loading limitations to the first stage. A minimum of two stages is required.

(e) Design Loading

The typical design loadings for non-nitrifying rotating biological contactors are shown in Table 5.3.

TABLE 5.3 **TYPICAL RBC DESIGN LOADING**	
Organic loading kg CBOD/d/10^3 m^2 of disc surface kg TBOD/d/10^3 m^2 of disc surface	 4 to 10 10 to 17
Maximum organic loading in the first stage kg CBOD/d/10^3 m^2 of disc surface kg TBOD/d/10^3 m^2 of disc surface	 20 to 30 40 to 60
Hydraulic Loading, m^3/d/m^2 of disc surface	0.02 to 0.08

(f) Enclosures

Enclosures should be provided for the RBC media to prevent algal growth on the media and minimize the effect of cold weather. Enclosures may be either fabricated individual enclosures or building enclosing several shafts. A building, enclosing the units, is preferable to individual enclosures, due to problems of any repair of the individual enclosures in the winter.

Individual enclosures, if proposed, should be made of material resistant to damage from humidity and corrosion. The exterior of enclosures should be resistant to deterioration from direct sunlight. Access points should be provided at each end of the enclosure to permit inspection of shafts and to perform operation and maintenance.

Enclosures should be removable to allow removal of the shaft assemblies. Access around enclosures should be sufficient to permit suitable lifting equipment access to lift covers and shafts.

Buildings should be designed with provision to remove shafts without damage to the structure. Buildings should also be designed with adequate ventilation and humidity control to ensure adequate oxygen is available for the RBC shafts, provide a safe environment for operating staff to perform normal operation and maintenance and minimize the damage to the structure and equipment from excess moisture. Building material and components should be resistant to corrosion.

(g) Hydraulics

The RBC design should incorporate sufficient hydraulic controls, such as weirs, to ensure that the flow is distributed evenly to parallel units. RBC tank design should provide a means for distributing the influent flow evenly across each RBC shaft. Intermediate baffles placed between treatment stages in the RBC system should be designed to minimize solids deposition. The RBC units should be designed with flexibility for series and parallel operation.

(h) Dewatering

The design should provide for dewatering of RBC tanks.

(i) Shaft Drives

The electric motor and gear reducer should be located to prevent contact with the wastewater at peak flow rates. Variable speed drives should be provided.

(j) Recycle

Effluent recycle after clarification should be provided for small installations where minimum diurnal flows may be very small. Recycle should be considered in any size plant where minimum flows are less than 30 percent of the average daily design flow.

(k) Load Cells

Load cells should be provided for each shaft. A clean water wet load should be derived at startup to provide for biomass measurement after growth occurs.

4. Aeration

(i) Oxygen Requirements

Secondary biological systems should generally be designed to supply oxygen to satisfy the carbonaceous biochemical oxygen demand. However, depending on the local conditions, the designer should also take into account the nitrogenous biochemical oxygen demand and inorganic chemical oxygen demand in calculating the oxygen requirements.

It is likely that nitrification (oxidation of ammonia nitrogen to nitrate nitrogen) will occur during the summer when temperatures are higher; without adequate oxygen, the onset of nitrification can lead to septic conditions and process upsets. Though nitrification is controlled during the summer period by reducing the inventory of MLSS in the aeration tank, some nitrogenous oxygen demand is inevitable and the designer is well advised to provide an allowance for oxygen transfer capability to satisfy these periods of partial nitrification.

Based on the wastewater characteristics, inorganic chemical oxygen demand should also be considered in calculating the oxygen requirements. For instance, hydrogen sulphide will have a demand on oxygen under septic conditions. The designer should evaluate the effect of wastewater septicity and include an allowance for oxygen requirements associated with hydrogen sulphide in the influent wastewater.

(a) Carbonaceous Biochemical Oxygen Demand

The carbonaceous oxygen demand may be estimated based on the process oxygen balance using the following formula.

$$R_c = Q\,(S_o - S)(1 + b\,O_c - BY)/(1 + bO_c)$$

where:

R_c = mass oxygen required per unit time to satisfy the carbonaceous biochemical oxygen demand.

Q = flow rate

S_o, S = total carbonaceous oxygen demand of the influent and effluent. The effluent CBOD should be assumed to be zero in the sizing of aeration equipment.

b = endogenous decay coefficient

Y = true cell yield

O_c = solids retention time

B = oxygen equivalent of cell mass

As an alternative, the carbonaceous oxygen requirements based on different sludge age are suggested below:

SRT	Oxygen Required (kg O_2 / kg CBOD)
5	1.0
10	1.1
15	1.2
20+	1.3

(b) Nitrogenous Biochemical Oxygen Demand

The nitrogenous oxygen demand may be estimated by the following equation:

$$R_n = 4.6 \times Q \times (N_o - N)$$

where N_o and N are influent and effluent oxidizable nitrogen respectively. The effluent nitrogen level should be assumed to be zero in the sizing of the aeration equipment.

(c) Inorganic Chemical Oxygen Demand

This oxygen demand is most often estimated based on a stoichiometric calculation.

(d) Spatial and temporal variations

In addition to the total oxygen demand caused by the above sources, the spatial and temporal variations in the demands within the reactor should also be considered in sizing the aeration equipment.

(e) Mixing requirement

In addition to oxygen transfer, sufficient mixing should be maintained such that the biological solids are kept in close contact with the wastewater as shown.

TABLE 5.4 MIXING REQUIREMENTS

Type of Equipment	Minimum Mixing Energy
Fine bubble diffused aeration	40 m^3/min/10^3.m^3 floor coverage
Coarse bubble diffused aeration	20 m^3/min/10^3.m^2 floor coverage
Mechanical	20 kW/10^3.m^3

(ii) Aeration System Alternatives

The aeration equipment can be classified under two categories. The first category is the diffused aeration system. This system supplies oxygen by introducing air into the wastewater with submerged diffusers or other aeration devices. The equipment under this category can be further divided into three groups: porous diffuser system, nonporous diffuser system and other aeration devices such as aspirators or jet aerators.

The second category is the mechanical aeration system, which supplies oxygen by agitating the wastewater mechanically so as to promote solution of air from the atmosphere. Mechanical aerators are usually divided into two major groups: aerators with a vertical axis and aerators with a horizontal axis. Both groups are further subdivided into surface and submerged aerators.

The selection of the aeration equipment should be based on both cost and non-monetary considerations. Under cold climatic conditions, special care should be given to freeze protection if surface aerators are proposed.

(a) Diffused Air Systems

Air volume requirements for channel, pumps, or other air-use demands should be added to the oxygen requirements of the activated sludge process.

The specified capacity of blowers, particularly centrifugal blowers, should take into account that the air intake temperature might reach extremes and that pressure might be less than normal. Motor horsepower should be sufficient to handle the minimum and maximum ambient temperature. Piping head loss must also be accounted for.

The blowers should be provided in multiple units, arranged and in capacities to meet the maximum air demand with the largest unit out of service. The design should also provide for varying the volume of air delivered in proportion to the load demand of the plant.

The diffusers should be spaced in accordance with the oxygen and mixing requirements in the basin and should be designed to facilitate adjustments of their spacing without major revision to air header piping. The arrangement of the diffusers should also permit their removal for inspection, maintenance, and replacement without dewatering the tank and without shutting off the air supply to other diffusers in the tank. Slip fittings should not be used. Pipe vibrations should be dampened.

Individual units of diffusers should be equipped with control valves, preferably with indicator markings for throttling and complete shutoff. Diffusers in each assembly should have substantially uniform pressure loss.

Flow meters and throttling valves should be placed in each header. Air filters should be provided to prevent clogging of the diffuser system.

For further details, the designer should refer to the USEPA publication entitled Fine Pore Diffused Aeration Manual.

(b) Mechanical Aeration Systems

The mechanism and drive unit should be designed for the expected conditions in the aeration tank in terms of the proven performance of the equipment.

Due to high heat loss, consideration should be given to protecting subsequent treatment units from freezing where it is deemed necessary. Multiple mechanical aeration unit installations should be designed to meet the maximum oxygen demand with the largest unit out of service. The design should normally also provide for varying the amount of oxygen transferred in proportion to the load demand on the plant.

A spare aeration mechanism should be furnished for single-unit installations.

(iii) Flexibility and Energy Conservation

The design of aeration systems should provide adequate flexibility to vary the oxygen transfer capability and power consumption in relation to oxygen demands. Particular attention should be given to initial operation when oxygen demands may be significantly less than the design oxygen demand. The design should always maintain the minimum mixing levels; mixing may control power requirements at low oxygen demands.

Dissolved oxygen probes and recording should be considered for all activated sludge designs. For larger plants, consideration should be given to automatic control of aeration system oxygen transfer, based on aeration basin dissolved oxygen concentration.

Watt-hour meters should be provided for all aeration system drives to record power usage.

Energy conservation measures should be considered in design of aeration systems. For diffused aeration systems, the following should be considered:

- use of smaller compressors and more units
- variable-speed drives on positive-displacement compressors
- intake throttling on centrifugal compressors
- use of timers (minimum mixing should be maintained)
- use of high-efficiency diffusers

For mechanical aeration systems, the following should be considered:

- use of smaller aerators
- variable aeration tank weirs
- multiple-speed motors
- use of timers.

5.3.1.7 Tertiary Treatment

1. General

The objective of the tertiary treatment (mechanical) is to achieve the effluent standards as specified in section 3.1.2, Table 3.2. The tertiary treatment entails nutrient (phosphorus and ammonia) control and effluent disinfection in addition to the reduction of carbonaceous biochemical oxygen demand and total suspended solids. The selection of the most appropriate process is dependent on the following factors:

- existing process, if any
- wastewater characteristics
- space and hydraulic constraints
- operator's preference.

2. Phosphorus control

Phosphorus control can be achieved biologically or chemically. The selection of the most appropriate control method should be made based monetary and process considerations. The important factors which impact on the costs comparison include: (1) influent phosphorus levels and loadings, (2) chemical costs, (3) sludge disposal costs, and (4) amenability of Biological Nutrient Removal (BNR) retrofit of the existing process. In addition to the monetary comparison, there are process advantages associated with the BNR process which should be considered:

- improved sludge settleability
- recovery of alkalinity (for denitrification).

(a) Biological Phosphorus Removal (BPR)

Biological Phosphorus Removal is accomplished by a group of organisms that have the ability to uptake quantities of phosphorus in excess of their synthesis requirements when stressed by environmental conditions. This is termed "luxury uptake" and occurs when anaerobic conditions are present in the influent region of a plug flow reactor and aerobic conditions elsewhere. Under these conditions, the phosphorus content of the mixed liquor increases from approximately 1.5 percent to as high as seven (7) percent solids. Phosphorus is removed via wasting sludge. The luxury uptake phenomenon is actually part of an energy storage cycle that allows the phosphorus accumulating organisms to become active in the anaerobic zone.

There are several factors that can affect the phosphorus removal efficiency of the BPR systems. These factors relate to wastewater characteristics, system design and operational methods. These factors can be divided into the following: (i) environmental factors, such as temperature, D.O. and pH, (ii) substrate availability as affected by influent wastewater characteristics, the level of volatile fatty acid (VFA) production, and the presence of nitrates, (iii) design parameters, such as system Sludge Retention Time (SRT), anaerobic zone detention time, and aerobic zone detention time, and (iv) effluent total suspended solids concentration.

There are various process configurations to achieve BPR. The process configurations include A/O, A2/O, Modified Bardenpho, UCT, VIP, Step Bio-P, PhoStrip and Bio-denipho and Trio-denipho processes. Some processes are proprietary. Most processes require different degrees of plant capacity derating to achieve BPR, when retrofitting existing wastewater treatment plants.

It is not the intention of this section to provide step-by-step guidelines for the design of a BNR system. However, adequate design consideration should be given to the following:

- account for the effect of sidestream returns
- avoid trapping of Nocardia/foam in the bioreactors
- provide flexibility of zonal hydraulic retention times (HRTs)
- provide good D.O. control of all aerobic cells
- include by-pass capability to reduce unnecessary tankage during initial or low load conditions
- provide flexibility of recycle sources and destinations
- pay careful attention to mixing energy and mixer placement
- avoid secondary release of phosphorus (release without energy uptake).

For additional details, the designer should refer to the USEPA publication entitled Design Manual on Nitrogen Control and Phosphorus Removal.

(b) Chemical Phosphorus removal

Phosphorus in wastewater can be removed chemically by the addition of metallic salts or lime. Lime addition is seldom practiced today due to the high chemical usage, problems associated with handling lime and the large volume of sludge generated from lime addition.

The metallic salts commonly used for phosphorus removal are aluminum or iron salts. Iron salts are less common for use in Alberta because of their costs, and limited availability. The common aluminum salts for phosphorus removal include alum, sodium aluminate, polyaluminum chloride and Polyaluminum Silicate Sulphate. Generally, the best removal result is achieved when an organic polymer is added with the metallic salts to assist the precipitation process. Jar tests should be carried out to determine the most appropriate chemicals or their combination and the optimal dosage.

The metallic salts and polymer can be added before the primary clarifier, before or inside the aeration basin and before the final clarifiers. It is a good practice to provide multiple feed points to improve process flexibility. The locations of the feed points should be chosen such that the flow conditions are turbulent to promote good mixing of chemicals.

3. Ammonia Removal

Ammonia can be removed either biologically or chemically using air stripping or breakpoint chlorination. Chemical method for ammonia removal is generally less cost effective and should be considered only under special cases.

The biological removal of ammonia is achieved by the biochemical oxidation of ammonia to nitrate with nitrite as an intermediate. Two autotrophic microorganisms, Nitrosomonas and Nitrobacter are responsible for these reactions. These bacteria grow slower than the BOD removal bacteria and are sensitive to low temperature and adverse environmental conditions. The most important requirement for achieving consistent nitrification is to maintain adequate sludge retention times to prevent these slow growing bacteria from washing out from the biological systems.

Alkalinity is consumed in the nitrification process (7.4 mg of alkalinity as $CaCO_3$ for every mg of NH_4-N nitrified). This loss of alkalinity may have an impact on the design of nitrifying plants treating poorly buffered or low alkalinity wastewater, because the nitrifiers also tend to be pH dependent.

Nitrification can be achieved by either a suspended growth or fixed film process. By incorporating dentrification into the design, approximately two thirds of the oxygen required for nitrification can be recovered.

4. Effluent Disinfection

(i) General

(a) Methods of Disinfection

Bactericides, Viricides and Potential Disinfectants

UV radiation, chlorine and bromine chloride may be used in full-scale plants to reduce microorganisms (bacteria and viruses) in the wastewater effluent.

Disinfectants

Ozone and chlorine dioxide may be used in full-scale plants to reduce microorganisms (cysts, bacteria, and viruses) in the wastewater effluent.

(b) Selection Method

The selection of a disinfection method should be based on both monetary and non-monetary considerations. In conducting cost comparison, capital, O&M and total life-cycle costs should be estimated. The non-monetary considerations should include disinfection effectiveness, state of development, effluent quality impacts, chemical hazards and safety concerns, process complexity and ease of operation and maintenance.

(ii) UV Radiation

(a) General

UV radiation may be used to achieve the disinfection requirements to produce effluent that meets Alberta surface water bacteriological quality for recreational waters. At the UV dose commonly applied, this process is effective in inactivating indicator bacteria such as those members of the coliform group, and to a lesser extent the viruses. At this dose, UV is not effective in inactivating protozoa.

(b) Types of UV Systems

UV disinfection systems can be broadly classified under three groups based on the types of lamps used: (A) low-pressure, low-intensity, (B) low-pressure, high-intensity, and (C) medium-pressure, high-intensity UV systems. The sources of UV radiation of these UV systems are mercury vapour lamps, which are operated at different mercury vapour pressures and discharge currents.

Low-pressure, high-intensity systems are uncommon and not widely used.

i) Low-Pressure, Low-Intensity Systems

Of the disinfection systems using low-pressure, low-intensity mercury lamps, there are three major different reactor designs: open channel, closed chamber and Teflon tube. In the open channel systems, the lamps can be oriented either horizontal and parallel-to-flow or vertical. Other reactor configurations such as closed chamber or Teflon tube should be considered under special cases only.

ii) Medium Pressure, High Intensity Systems

There are three designs using this type of lamps: horizontal and parallel-to-flow, horizontal and perpendicular-to-flow or closed chamber.

(c) Selection of UV Disinfection Alternatives

The factors which affect the selection of the appropriate UV system include:

- design flow rate
- wastewater transmittance and suspended solids levels
- effluent bacteria standards

- continuous or seasonal disinfection
- lamp fouling potential
- land and hydraulic constraints
- power charges
- labour costs
- headloss constraints.

Generally, a low-pressure, low-intensity system is suitable for small installations (less than 300 lamps). For major installations, detailed cost comparison of the promising alternatives should be conducted before the UV system is selected.

(d) Influent Characterization

The design of a UV disinfection facility requires a thorough understanding of the characteristics of the influent to the UV facility. The important parameters include flow rates, UV transmittance, total suspended solids and coliform levels. These parameters have direct impact on the UV disinfection efficiency. Other parameters which should also be considered include BOD, ammonia, nitrate, iron, hardness, oil and grease and wastewater temperature. They either have indirect impact on disinfection efficiency or direct impact on lamp fouling.

When a UV disinfection facility is added to an existing treatment plant, the historical data regarding effluent qualities should be studied. Statistical and probabilistic analyses should be conducted to establish the expected worst-case operating conditions as the design basis. Measurement of UV transmittance levels should also be conducted.

If UV disinfection is proposed for a new plant, the effluent qualities of similar installations and available published information may be used as the design basis; or pilot plant studies should be considered. The presence and nature of any industrial discharge and the raw water source and quality should be considered. The effluent UV transmittance depends on the type and degree of treatment. Generally, a higher degree of treatment gives a higher light transmittance of the effluent. The UV transmittance from a suspended growth process is generally higher than that from a fixed film process. Typical ranges of UV transmittance values for various secondary processes are shown in Table 5.5.

TABLE 5.5
RANGE OF UNFILTERED UV TRANSMITTANCE

Treatment Processes	Typical Range
Conventional activated sludge	35 to 65 percent
Fixed film process	30 to 50 percent
Lagoon	30 to 50 percent
Secondary plus filtration	50 to 70 percent

(e) Assessment of Minimum UV Dose Requirements

UV dose is defined as the product of the intensity of radiation (microwatts per square centimeter) and the length of time (seconds) during which the wastewater is exposed to UV radiation. The minimum UV dose requirement is dependent on the design influent characteristics and the effluent coliform standards.

The minimum UV dose requirements may be determined using the following methods:

- *Pilot Testing*

  Pilot testing should be conducted if (A) the wastewater treatment plant (WWTP) is treating significant industrial wastes, (B) it is designed for major installations, or (C) a less common UV system is considered. The pilot-testing programme should include an intensive test programme to develop the dose response relationship of the UV system and a routine test program to assess the lamp sleeve fouling potential.

- *Disinfectability Studies*

  The disinfectability of the wastewater may be assessed in a laboratory using a collimated beam test apparatus. The results may be used to develop the dose response relationship under ideal laboratory conditions. Considerations should be given to allow for the less perfect hydraulic design of the full-scale UV reactor.

- *Mathematical Models*

  The minimum dose requirements may be determined using mathematical models such as those developed by USEPA or WERF. When properly calibrated using the data collected from the pilot studies, these models will provide a useful means for estimating the required dose under the design influent conditions. If the models are not calibrated using the actual wastewater characteristics, adequate safety margin should be allowed in the design.

- *Design and Operating Data from Similar Installations*

  The design and historical operating data of some existing UV installations may be used as the design basis. The data to be used in analyses are the influent and effluent coliform data, TSS levels, UV transmittance and flow rates. Lamp cleaning frequency records should also be reviewed. The important criteria for selecting the similar installations include the degree of treatment, the types of treatment processes and industrial components of the wastewater.

(f) Development of Design Dose and Lamp Requirements

The UV dose used to design the UV facility should be adjusted to account for aging and fouling effects of the UV lamps. The minimum UV dose developed based on clean and new lamps should be increased by the following formula:

Design UV dose = minimum UV dose/Fp/Ft

where F_p = the lamp output reduction factor. This factor is the fraction of initial output the lamp is expected to have at the end of its useful life.

F_t = the quartz sleeve transparency reduction factor. This factor is the fraction of lamp emission that is transmitted through the quartz sleeve immediately before the sleeve is cleaned. It includes the UV output loss due to the clean quartz sleeve and the losses due to the deposits on the sleeve (fouling).

Table 5.6 shows the suggested correction factors for the low-pressure UV systems. The appropriate correction factors for the high intensity UV systems should be decided from the full- or pilot-scale operating data.

TABLE 5.6
DESIGN CORRECTION FACTORS

UV Systems	F_p	F_t(1)
Low-pressure, low-intensity	0.65	0.71

Note:
The power correction due to loss in transmittance is based on clean quartz sleeve loss of 11 percent and the transmittance losses due to sleeve fouling of 20 percent for low-pressure systems.

The lamp requirements of the UV facility should be calculated based on the design UV dose and the design peak flow rate. Standby UV disinfection capacity is generally not required at peak flow conditions to account for units out of service. UV disinfection equipment cleaning or maintenance procedures that require removal of equipment from service should be able to be completed during the expected low flow conditions. Where peak flows are frequent or unpredictable, or the installations only have limited lamps, standby equipment during peak flow conditions should be provided.

(g) Lamp Arrangements

UV lamps are generally grouped in units of modules. The modules are then grouped into lamp banks. Multiple lamp banks are placed in a UV channel and the complete UV facility consists of one or more UV channels.

The choice of UV lamp numbers in a module is generally limited by the design of the UV system. The number of UV lamps in a low-pressure, low-intensity system of horizontal lamp design may range from two to 16 lamps. The number of UV lamps in a low-pressure, low-intensity system of vertical lamp design is typically 40. If a 16 horizontal lamp module or a 40 vertical lamp module is proposed, a mechanical lifting device should be installed to facilitate lamp removal and installation.

The numbers of UV modules in a UV bank are generally determined by hydraulic considerations. The appropriate number of UV modules (and number of lamps per module for horizontal lamp system) should be selected so as to ensure that the UV reactor is conducive to plug flow.

The number of UV banks in a channel should be at least two or four for horizontal and vertical lamp systems, respectively to maintain plug flow characteristics and minimize short-circuiting. For low-pressure, low-intensity systems, two UV channels should be provided to maintain system reliability.

In designing the lamp arrangement, a main important consideration is to facilitate flow pacing. The lamps should be arranged in such a way that the lamp banks can be turned on and off easily to match the expected disinfection needs. Some newer vertical-lamp systems are designed such that individual rows of lamps within a given module may be turned on and off based on flow rate signals.

(h) Reactor Designs

- *Inlet and Outlet*

  UV disinfection reactors should be designed with inlet channel approach and outlet conditions that promote plug flow within the system. To ensure proper inlet and outlet flow conditions, the following criteria are suggested:

  - Unobstructed approach channel length before first UV bank not less than 2 times channel water depth or 1.2 m.
  - Unobstructed downstream channel length following last UV bank before water level control device not less than two (2) times channel water depth or 1.2 m.
  - Spacing between UV banks = minimum spacing required for maintenance and access

- *Flow Distribution*

  If more than one UV channel is provided, a positive flow distribution system such as weirs should be used to ensure equal flow splitting. The influent chamber should be sized such that the headloss along the chamber is less than one tenth of the headloss at each channel under the expected range of flow conditions.

- *Velocity Distribution*

  For low-pressure, low-intensity system, perforated stilling plates should be provided upstream of the first lamp UV lamp banks to ensure uniform velocity distribution.

- *Water Level Control*

  Water depth control devices should be provided in the low pressure, low-intensity systems to maintain a constant water level (plus or minus 2.5 cm) under the expected flow ranges. Weighted adjustable flap gates are generally suitable. If the flow rates during the initial years of operation are significantly lower than the design flow rates, a fixed weir should be installed initially. The water depth control for a medium-pressure, high-intensity system is less critical because its main function is to ensure full submergence of the reactors at all times. Weighted adjustable flap gates and adjustable weir gates are acceptable.

- *Isolation Gates*

  Sluice or weir gates should be provided upstream and downstream of the UV banks to isolate the UV channel when maintenance is needed. Flap gates which may leak under low flow conditions should not be used as isolation gates.

- *Drainage*

  A drainage (mud) valve should be provided in each UV channel for dewatering.

- *Hydraulics*

  Adequate hydraulic head should be allowed for in the design of the UV facility. The main hydraulic head losses occur at the inlet for flow splitting and at the outlet gates for free flow discharge. The losses at the UV banks and reactors are generally small when compared with other losses.

- *Flood Level*

  The top level of the UV process area should be located above the 1:100 year flood level. The channel depths should be checked such that the UV modules can be conveniently lifted up from the main process area.

(i) Lamp Cleaning

Adequate facilities should be provided to facilitate cleaning of lamp sleeves. In addition to the cleaning chemical system, appropriate wash down area and lifting devices should be provided.

For low-pressure, low-intensity systems, lamp cleaning can be performed manually or with a dip tank for small installations. In large installations, a cleaning chemical (acid) bath should be provided so that one lamp bank can be cleaned at a time. If in-channel lamp cleaning system is proposed, suitable concrete lining and isolation gates should be provided. Gates should preferably be of stainless steel or fibreglass.

For medium-pressure, high-intensity systems, cleaning of lamps may be performed by the built-in wiping mechanisms, as included in some designs. If only a mechanical wiper is provided, provision should be made for cleaning the lamps chemically on a regular basis.

(j) Screens

Screens may be required immediately upstream of the UV facilities to remove algal clump or other objects which may impact UV disinfection performance, or damage the lamps. Screen openings may range from 2 to 13 mm, depending on the selected types of UV systems, size of installations, the expected degree of algae problem and availability of operating resources. Mechanical screens should be considered for large installations.

(k) Ballasts

The ballasts should be compatible with the proposed lamp type. If possible, the more energy efficient electronic ballast should be specified.

(l) Control and Instrumentation

The choice of the most appropriate control system for a UV facility is dependent on the size of the installation, the available operating staff and the control system for other WWTP facilities. For major installations, the functions of a UV disinfection control system should include the following:

- activate and deactivate UV banks and channels based on the disinfection needs
- activate and deactivate lamp cleaning mechanism (for medium-pressure, high-intensity systems)
- monitor influent characteristics and equipment operation status
- generate alarms
- monitor UV intensity.

The major parameters to be monitored continuously in the control system may include flow rate, UV transmittance, UV intensity, water levels, UV output and lamp/bank status. Other important information for process control such as influent and effluent coliform levels and total suspended solids levels will be collected by taking samples manually.

For small installations, the minimum function of a control system should include flow pacing of the UV lamp banks to optimize the UV dose. While overdosing of UV radiation may not create any harmful environmental by-products, it will increase the O&M costs necessarily by wasting power and reducing useful lamp life.

(m) Housing

The need for housing of the UV facility should be decided on a case-by-case basis. Under extreme climatic conditions, housing is generally desirable for low-pressure systems where manual lamp cleaning is required.

If housing is provided, attention should be paid to humidity control and ease of equipment transport into and within the building.

(n) Sequencing Batch Reactor

If the UV disinfection is proposed with a sequencing batch reactor system, consideration should be paid to the intermittent effluent discharge. The design flow rates should be decided based on the decant flow rates. If decant system is not designed to have constant-flow design, the maximum flow rate during the decant cycle should be used to size the UV disinfection. It may be appropriate to provide an upstream equalization basin to provide continuous flow through the UV system.

The number of expected on-off operating cycles of the UV lamp banks should be estimated. If the expected number of on and off is high, provisions should be made of an equalization basin upstream of the UV facilities or recycle pumps downstream of the UV facilities.

(o) Safety

Three safety issues should be addressed fully in the design of a UV disinfection facility. They are: (1) exposure to UV radiation, (2) electrical hazards, and (3) handling of acids.

Exposure to UV radiation may affect the eyes with a temporary painful condition known as conjunctivitis or "welder's flash". Bare skin will also be burned upon exposure to UV at these wavelengths. For the low-pressure, low-intensity UV systems, eye shields should be provided in the UV channels. The use of chequer plate instead of open floor grating should also be considered. The medium-pressure, high-intensity systems must be completely enclosed. All UV systems must be equipped with safety interlocks that shut off operating modules if they are removed from the channel.

The safety requirements for handling high-voltage electricity should be followed. All the electrical components should be designed for submergence or located above the 1:100 flood levels. Electrical hazards should be minimized by the inclusion of ground fault interruption circuitry with each operating module.

Due considerations should be given to the cleaning chemical tank design such that the chance of accidental falling into the tank is minimized. Handrails should be provided when needed. Safety shower and eye wash facilities should be provided in the chemical bath and chemical handling areas.

(iii) Ozonation

(a) General

Ozonation is generally applied only to effluents that are nitrified, highly clarified (filtered) or both. This method may also be considered for unfiltered secondary effluents when a low-cost source of oxygen is available, e.g., at a pure oxygen activated sludge WWTP.

(b) Source of Ozone

Ozone must be generated on site because it is chemically unstable and decomposes rapidly to oxygen after generation. The most efficient method of producing ozone is by electrical discharge using either air or pure oxygen.

(c) Design Requirements

The design requirements for ozonation systems should be decided based on pilot testing or similar full-scale installations. As a minimum, the following design factors should be considered:

- ozone dosage
- dispersion and mixing of ozone in wastewater
- contactor design
- control of off-gas.

(d) Gas Selection and Preparation

Ozone may be generated from air, oxygen-enriched air or high purity oxygen. The concentration of ozone produced increases 2 to 2.5 times when air is replaced by high-purity oxygen at the same gas flow rate. The selection of feed gas should be decided based on cost and other considerations. Generally, the use of pure oxygen is not economical unless high-purity oxygen is required elsewhere in the treatment facility.

Regardless of whether air or high purity oxygen is used as the feed gas, gas preparation is required to ensure it is free of oil, dust and moisture. When ambient air is used as the feed air, the following processing and control units should be provided prior to ozone generation:

- filter - 5 µm modular or fabric filter
- pressurizer - blower, compressor
- after cooler
- oil coalescer - if an oil-free pressurization device is not used
- refrigerant drier - to reduce the size of desiccant drier

- desiccant drier - silica gel, activated alumina, or crystalline zeolite
- filter - 99 percent efficient at 1.0 µm size
- hygrometer
- gas flow meter
- pressure release valve - in high-pressure systems.

When oxygen enriched air is used, the gas preparation units are similar to those required for ambient air except that the desiccant drier is not needed and replaced by a pressure swing separator.

In the case of high-purity oxygen, the following units should be provided prior to ozone generation:

- high-purity oxygen source -cryogenic plant, pressure swing absorption unit or oxygen cylinders
- pressurizer - blower, compressor
- after cooler
- oil coalescer - if an oil-free pressurization device is not used
- refrigerant drier - to reduce the size of desiccant drier
- desiccant drier for recycled oxygen and
- filter - 99 percent efficient at 1.0 µm size
- pressure release valve - in high-pressure systems.

(e) Ozone Generation

- *Types*

  The ozone generators may be broadly classified based on their power supply, i.e., low frequency (60 Hz), medium frequency (up to 600 Hz) and high frequency. These types may be further divided based on the cooling media (air, water or both) and the physical arrangements of the dielectrics.

  Small systems (less than 450 kg/d) generally call for the application of low frequency water-cooled units. In larger systems, especially those incorporating high purity oxygen process, the added cost and complexity of higher frequency generators with associated chilled water-cooling systems may be justified.

- *Sizing*

  The total capacity of the ozone generators should be decided such that the required applied ozone dosage can be delivered under peak flow condition. The applied ozone dosage should be calculated based on the required absorbed ozone dosage and the minimum ozone transfer efficiency at the expected ozone dosage and wastewater quality conditions.

  Standby ozone generator capacity is generally not required at peak conditions. Ozone equipment maintenance that requires removal of equipment from service should be able to be completed during expected low flow or dose requirement conditions. Where peak flows are frequent or unpredictable, stand-by equipment during peak flow condition should be considered.

  The number and size of generator units should be decided such that the system can satisfy both the maximum and minimum ozone production rates under the expected operation conditions.

(f) Ozone Contacting

The ozone-contacting basin should be designed such that there is an efficient mass transfer of ozone out of gas bubbles into the bulk liquid and sufficient time for disinfection. Common contactor types include:

- diffused bubble (concurrent and counter current)
- positive pressure injection
- negative pressure (venturi)
- mechanically agitated
- packed tower.

For diffused bubble systems, the contactor should be at least 6 m deep for secondary effluent at an applied ozone dosage of less than 6 mg/L and an elevation of approximately 1000 m in Alberta. The contactor may be deeper if the wastewater to be disinfected is of higher quality, if the applied ozone dosage is higher or if the plant is located at a higher elevation.

Multiple staged ozone contactors should be provided to minimize the effect of short-circuiting. A minimum of three and preferably more stages should be provided. Each stage should be positively isolated from the other to simulate plug flow characteristics and minimize the potential for short-circuiting. The minimum contact time should be six minutes and preferably 10 minutes at design flow rates.

The off-gases from the contact chamber must be treated to destroy any remaining ozone. The product formed by destruction of the remaining ozone is pure oxygen, which may be recycled if pure oxygen is being used to generate the ozone.

(g) Housing

If a building is not provided separately for ozone generation equipment, a gas-tight room should be constructed to separate the ozone equipment from other part of the building. Doors to the ozone generation room should open only from the outside of the building and should be equipped with panic hardware.

At least two means of exit should be provided from each separate room. All exit doors should open outward.

A clear glass, gas tight window should be installed in an exterior door or interior wall of the ozone generation room to permit the ozone generator to be viewed without entering the room.

(h) Ventilation

For ozonation system rooms, continuous mechanical ventilation should be provided to maintain at least six air changes per hour. The entrance to the air exhaust duct from the room should be near the floor and the point of discharge shall be selected such that it will not contaminate the air inlet of any buildings or inhabited areas.

(i) Corrosion Protection

The selection of material should be made with due consideration for ozone's corrosive nature. Only materials at least as corrosion-resistant to ozone as grade 304 L or 316 L stainless steel should be specified for piping containing ozone in non-submerged applications. Unplasterized PVC may be used in submerged piping, provided the gas temperature is below 60°C and the gas pressure is low.

Piping systems should be as simple as possible, specifically selected and manufactured to be suitable for ozone service, with a minimum number of joints. Piping should be well supported and protected against temperature extremes.

(j) Safety

The safety issues that should be addressed fully in the design of ozonation system include: (1) exposure to ozone, (2) noise, and (3) electrical hazards.

The occupational exposure of ozone should be controlled such that workers will not be exposed to ozone concentrations in excess of 0.2 mg/m^3 for eight (8) hours or more per workday, and that no worker be exposed to a ceiling concentration of ozone in excess of 0.6 mg/m^3 for more than 10 minutes.

All ozone systems should be provided with an ambient ozone monitor or monitors which are set up to measure the ozone concentration at potential ozone-contaminated locations within the plant. The monitors should be set up to sound an alarm when the ozone concentration reaches 0.2 mg/m^3 and should be set up to shut down the ozone system when the concentrations exceed 0.6 mg/m^3.

Eyewash basins should be provided to enable the operator to rinse ozone from the eyes, if needed. The basins should be located in the outside of the ozone generator room.

Some ozone generation systems are classified as noisy installations. Generally, the main source of noise is the feed-gas compressor. Wherever practical, the feed-gas compressors should be isolated in a sound insulated room.

The ozone generators generally require high power consumption. All safety requirements regarding handling of high voltage power should be followed.

(iv) Chlorine

(a) General

Chlorine is an effective disinfectant. Potential drawback to its use include:

- toxicity to aquatic, estuarine and marine organisms
- generation of harmful chlorinated disinfection by-products
- safety concerns during transportation, storage and handling, particularly with gaseous chlorine.

If chlorination is proposed, due consideration should be given to the above factors. A dechlorination system should be provided.

(b) Forms of Chlorine

Chlorine may be added to the wastewater in the form of liquid/gaseous chlorine or sodium hypochlorite.

(c) Design Requirements

In sizing a chlorination system, the following factors should be considered:

- contact time
- concentration and type of chlorine residual
- mixing
- pH

- suspended solids levels
- temperature
- coliform levels
- ammonia concentrations.

The design should provide adequate flexibility in the chlorination and control system to allow controlled chlorination at the expected flow ranges in the design period. Special consideration should be given to the chlorination requirements during the initial years of operation to ensure the chlorination system is operable at less than design flows without over-chlorination.

(d) Chlorine Addition

Chlorine should be added into the wastewater where good mixing is achieved at all times. Mixing may be accomplished mechanically or hydraulically.

When mechanical mixing is proposed, the following criteria apply:

- a mixer-react unit is necessary that provides 0.1 to 0.3 minutes contact
- chlorine should be injected just upstream of the mixer with a diffuser
- the minimum mixer speed should be 50 revolutions per minute
- the diffuser should be set at least two feet below the minimum wastewater level at low flows
- turbulent flow after complete chlorine mixing should be avoided to prevent chlorine stripping.

Hydraulic mixing should be achieved based on the following criteria:

- Pipe Flow
    - A Reynolds number of greater than or equal to 1.9×10^4 should be achieved at all flow rates. Hydraulic jumps or baffles may be used to create turbulence.
    - A diffuser, with orifice velocities of 5 m/s at peak flows should be provided.
    - The diffuser should be set as deep as possible and at least two feet below minimum wastewater level at low flows.

- Open Channel

  - A hydraulic jump with a minimum Froude number of 4.5 is necessary to provide the adequate hydraulic mixing. Multiple points of chlorine injection should be provided because the jump location may change with changes in flow rates. A parshall flume is not a satisfactory location for hydraulic mixing.

(e) Contact Basin

Contact chambers should be sized to provide sufficient retention time for the effluent to meet the required bacteriological quality.

The contact chambers should be baffled to minimize short-circuiting and back mixing. Baffles should be constructed parallel to the longitudinal axis of the chamber with a minimum length-to-width ratio of 40:1. Side water depths should range between 2 to 5 m.

(f) Dechlorination

Dechlorination may be achieved by the use of detention ponds or by the addition of sulphur dioxide or sodium metabisulphite to the chlorinated effluent.

The required sulphur dioxide dosage for dechlorination is 1 mg/L SO_2 for 1 mg/L chlorine residual. Reaction time is essentially instantaneous. Detention time requirements are decided based on the time necessary to ensure complete mixing of the sulphur dioxide. To ensure continuous compliance of the maximum chlorine residual requirements, over-dechlorination followed by re-aeration should be considered. Continuous monitoring of the effluent would be a requirement.

(g) Chlorination equipment and chlorine room design requirements

For details of chlorine equipment requirements and chlorine room design requirements, see section 1.5.3.1 (2) and (3).

5.3.1.8 Secondary Clarifier

Design of a secondary clarifier for suspended growth systems is different from the design of secondary clarifier for fixed growth systems, in that, to perform properly while producing a concentrated return flow, a suspended growth secondary clarifier should be designed to meet thickening as well as solids separation requirements. Since the rate of recirculation of return sludge from the final settling tanks to the aeration is quite high in activated sludge processes, surface overflow rate and weir overflow rate should be adjusted for the various processes to minimize problems with sludge loadings, density currents, inlet hydraulic turbulence, and occasional poor sludge settleability. The size of the settling tank should be based on the larger surface area determined for surface loading rate and solids loading rate.

Design criteria for secondary clarifier is detailed in Table 5.7.

TABLE 5.7
SECONDARY CLARIFIER DESIGN CRITERIA

Treatment Process	Surface Loading Rate at Peak Design Flow[1] ($L/s/m^2$)	Peak Solids Loading Rate[2] ($kg/d/m^2$)	Minimum Water Depth (m)	Weir Overflow Rate[3] (L/s/m)
RBC Trickling filters	0.56	-	3.0	2.9 to 4.3
Conventional-activated Sludge; Contact Stabilization	0.56*	245	3.7**	2.9 to 4.3
Extended Aeration	0.47	171	3.7	2.9 to 4.3

[1] Based on influent flow only. Lower loading rate should be used for nitrifying plants and those with chemical addition for phosphorus removal

[2] Clarifier peak solids loading rate should be computed based on the maximum day design flow plus the maximum return sludge rate requirement and the design mixed liquor suspended solids (MLSS) under aeration.

[3] Weir overflow rate would increase with increasing plant capacity.

* Plants needing to meet 20 mg/L suspended solids should reduce surface loading rate to 0.47 $L/s/m^2$.

** Greater water depths are recommended for clarifiers in excess of 372 m^2 surface area. Less than 3.7 m water depths may be adequate for package plants with average design flow less than 100 m^3/d.

5.3.1.9 Laboratory Requirements

All treatment works should include a laboratory for making the necessary analytical determination and operating control tests, except where satisfactory off-site laboratory provisions are made to meet the operating approval monitoring requirements. The laboratory should have sufficient size, adequate ventilation (particularly where furnace and fume hoods are used for solids and sludge analysis), bench space, equipment, and supplies to perform all self-monitoring analytical work required by the operating approval, and to perform the process control tests necessary for good management of each treatment process included in the design.

The laboratory arrangement should be sufficiently flexible to allow future expansion should more analytical work be needed. Laboratory instrumentation and size should reflect treatment plant size, staffing requirements, and process complexity. Experience and training of plant operators should also be assessed in determining treatment plant laboratory needs.

Before undertaking the detailed design of the laboratory facility, contact should be made with AENV to confirm the testing requirements.

5.3.1.10 Flow Measurements

1. Location

Flow measurement facilities should be provided to measure the following flows:

(i) plant influent and effluent flow;

(ii) plant and process unit bypasses; and

(iii) other flows such as return activated sludge, waste activated sludge, recirculation, and recycle required for plant operational control.

2. Facilities

Indicating, totalizing, and recording flow measurement devices should be provided for all mechanical plants. All flow measurement equipment must be sized to function effectively over the full range of flows expected and shall be protected against freezing.

3. Hydraulic Conditions

Flow measurement equipment including entrance and discharge conduit configuration and critical control elevations should be designed to ensure that the required hydraulic conditions necessary for accurate measurement are provided. Conditions that must be avoided include turbulence, eddy currents, air entrainment, etc. that upset the normal hydraulic conditions that are necessary for accurate flow measurement.

5.3.1.11 Colour Codes

Refer to Table 5.8 for recommended colour coding for wastewater treatment plant piping.

TABLE 5.8
RECOMMENDED COLOUR CODING FOR WASTEWATER TREATMENT PLANT PIPING

Piping to be Identified	Basic Colour	Bands	
		No.	Colour
Raw Wastewater	Brown	-	-
Primary Settled Wastewater Effluent	Brown	1	White
Secondary Settled Wastewater Effluent	Gray	-	-
Sludge Lines			
Raw Wastewater	Black	-	-
Primary Sludge	Black	1	White
Secondary Sludge	Black	2	White
Digested Sludge	Black	3	White
Digested Liquor	Black	1	Brown
Natural Gas	Orange	-	-
Digester Gas	Orange	1	Black
Chlorine Gas	Yellow	-	-
Chlorine and Water	Pink	1	Yellow
Chlorinated Effluent	Grey	1	Yellow
Electrical	Purple	-	-
Compressed Air	White	-	-
Heating	Silver	-	-
Fire Protection	Red	-	-
Potable Water	Blue	-	-
Untreated Water	Dark Green	-	-

Notes

1. Entire length of pipe to be painted in basic colour.
2. Bands, if required, are to be placed as follows:
   (a) at 9 m intervals, and/or
   (b) where the pipe enters and leaves a room.
3. Individual bands are to be 25 mm wide, and a 25 mm space is to be left between bands where multiple bands are required.

5.3.2 Aerated Lagoons

5.3.2.1 General

The aerated lagoon system is a biological treatment process with long retention time (compared to mechanical systems) and large capacity. The system consists of one or more "complete mix cell" and one or more "aerated cell."

The complete mix cell is designed to provide enough oxygen transfer to satisfy the applied CBOD loading and to maintain a uniform solids concentration. The aerated cell is designed to satisfy the applied CBOD loading while maintaining an adequate uniform dissolved oxygen level in the cell. There is no attempt in the design of the aerated cell to provide complete solids mixing, therefore solids are allowed to settle in the cells to undergo anaerobic decomposition.

5.3.2.2 Design Approach

In general, the following factors should be considered in the design of the aerated lagoons:

- CBOD removal and effluent characteristics;
- Temperature effects;
- Mixing requirements;
- Oxygen requirements; and
- Solids separation.

1. **CBOD Removal and Effluent Characteristics**

CBOD removal and the effluent characteristics are estimated using a complete mix hydraulic model and first order reaction kinetics. The complete mixed model using first order kinetics and operating in a series with 'n' equal volume cells is given by:

$$\frac{L_e}{L_i} = \frac{1}{\left[1 + \frac{K_t T}{n}\right]^n}$$

where:

L_e = Effluent BOD, mg/L
L_i = Influent BOD, mg/L
K_t = Reaction rate coefficient at t°C, day^{-1}
T = Total hydraulic retention time in lagoon system, days
n = Number of ponds in series

The selection of the reaction rate coefficient is critical in the design of the lagoon system. All other considerations in the design will be influenced by this selection. If possible, a design K_{20} should be determined for the wastewater in pilot or bench scale tests; experiences of others with similar wastewaters and environmental conditions should also be evaluated. Reaction rate coefficient K_{20} may vary from 1.5 day^{-1} for complete mix cell to 0.37 day^{-1} for aerated cell.

When using the complete mix model, the number of cells in series has a pronounced effect on the size of the aerated cell required to achieve a specific degree of treatment.

The reactor required to achieve a given efficiency may be greatly reduced by increasing the number of cells in series.

2. Temperature Effects

The influence of temperature on the reaction rate is expressed by the equation:

$$K_t = K_{20}\,\theta^{t-20}$$

where:

K_t = Reaction rate coefficient at t°C, day^{-1}
K_{20} = Reaction rate coefficient at 20°C, day^{-1}
t = Wastewater temperature, °C
θ = Temperature activity coefficient (varies from 1.04 to 1.1 for aerated lagoons, with typical value of 1.065)

3. Mixing Requirements

Aeration is used to mix the pond contents and to transfer oxygen to the liquid. There is no rational method available to predict the power input necessary to keep the solids suspended. The best approach is to consult equipment manufacturers' charts and tables to determine the power input needed to satisfy mixing requirements. Power of 6-10 w/m^3 of the cell volume is frequently used and these values can be used as a guide to make preliminary estimates of power requirements, but the final sizing of aeration equipment should be based on guaranteed performance by an equipment manufacturer.

For a complete mix cell, in comparing the power requirements for both, to maintain solids in suspension and to meet the oxygen demand, it would soon become evident that the mixing requirements would control the power input to the system.

After determining the total power requirements for a cell, the diffusers/aeration units should be located in the cell so that there is an overlap of the diameter of influence providing complete mixing.

4. Oxygen Requirements

Oxygen requirements are estimated using equations based upon mass balances, and there are several rational equations available to estimate the oxygen requirements for lagoon systems, however, the use of the CBOD entering the pond as a basis to estimate the biological oxygen requirements, is as effective. Approximately 1.5 kg to 2.0 kg of oxygen is required to remove 1 kg of CBOD in the aerated lagoon system.

As for the other mechanical systems, monitoring of aerated lagoons include adjustment of aeration devices to control dissolved oxygen to be greater than 2 mg/L in the aeration basin during peak loading conditions. Lagoons with odour problems should be corrected by increasing the air supply.

5. Solids Separation

For systems with continuous discharge to a receiving stream, a polishing cell having a minimum hydraulic retention of five days, based on summer average daily design flows, should be provided. Polishing cells are not required for systems having storage facilities with intermittent discharges.

5.3.2.3 System Design (Aeration)

The preceding section outlined the general design approach for designing aerated lagoon systems. This section outlines the procedure for the determining the aeration requirement for mixing (Step 1) as well as for satisfying the applied CBOD loading (Steps 2 to 6) in a step format.

Step 1: Determine aeration requirement for mixing in the complete mix cell. As indicated earlier, a power input of 6 - 10 w/m^3 per cell volume may be assumed to make the preliminary estimates, however the final capacity should be based on guaranteed performance by an equipment manufacturer.

Step 2 - Calculate the CBOD reduction in each cell under both summer and winter conditions.

Modifying the formula under section 5.3.2.2(1) for a single cell, CBOD reduction (complete mix cell) as a % of the average daily design CBOD load (E_1);

$$E_1 = \frac{K_t T}{1 + K_t T} \times 100$$

CBOD reduction (aerated cell) as a % of the average daily design CBOD load (E_2);

$$E_2 = \frac{K_t}{1 + K_t T} \times E_1 \times 100$$

CBOD reduction (overall) $E_3 = E_1 + E_2$

K_t values can be determined utilizing the formula noted in section 5.3.2.2(2).

Insert the calculated % reduction values on line 1 in the summary Table 5.8.

Step 3 - Calculate the CBOD removal (Kg) in each cell during both summer and winter conditions as follows:

CBOD removed in complete mix cell = average daily design CBOD load x E_1 (summer and winter).

CBOD removed in Aerated Cell = average daily design CBOD load x E_2 (summer and winter).

Insert the calculated CBOD removed in each cell on line 2 in the summary Table 5.8 and complete line 3.

Step 4 - Calculate the Actual Oxygen Required (AOR) in Kg/hour. This is determined by multiplying the calculated CBOD removal rate as noted on line 3 in the summary table by the oxygen to CBOD ratio (oxygen required in Kg per Kg of CBOD removed). As indicated in the foreceding section, an oxygen to CBOD ratio of 1.5 to 2.0 could be used when designing a system handling typical domestic wastewater.

Insert the AOR on line 4 in the summary Table 5.8.

Step 5 - Calculate the Standard Oxygen Required (SOR) in Kg/hour for each cell and for the total system during summer and winter conditions. This requires determination of the oxygen mass transfer ratio (φ) which is utilized to correct the rate of oxygen transfer under standard conditions to the rate of oxygen transfer to the wastewater under site conditions. The formula for determining the oxygen mass transfer ratio is as follows:

$$\varphi = \frac{\alpha(\beta C_s - C_L)\gamma^{t-20}}{C_s}$$

where:

φ = oxygen mass transfer ratio

α = correction factor used to estimate the actual oxygen transfer in wastewater versus the oxygen transfer in low total dissolved solids water generally used for rating aeration devices. α values of 0.6 to 0.8 are generally used for design.

β = correction factor used to correct the test system oxygen transfer rate for differences in oxygen solubility due to constituents in the wastewater such as salts, particulates and surface active substances. Values range from 0.7 to 0.98. A value of 0.95 is commonly used in the absence of experimental verification.

γ = factor for correcting the oxygen mass transfer rate at temperatures other than 20°C. Typical values are in the range of 1.015 to 1.040. A value of 1.024 is typical for both diffused and mechanical aeration devices.

C_s = solubility of oxygen at site conditions (i.e., site temperatures and barometric pressure) - mg/l.

C_L = minimum required dissolved oxygen concentration in the treated effluent - (2 mg/L during peak loading conditions).

t = wastewater temperature °C

Step 6 - Calculate the SOR required in Kg/hr for each cell and for the total system under both winter and summer conditions as follows:

SOR for each cell = AOR/φ

Total SOR = sum of the SOR for all cells

Insert the SOR values on line 6 in the summary Table 5. 9.

TABLE 5.9
AERATION SYSTEM DESIGN
SUMMARY TABLE

Line No.	**Summer**			**Winter**		
	Mix Cell	**Aerated Cell(s)**	**Total**	**Mix Cell**	**Aerated Cell(s)**	**Total**
1. CBOD reduction as a % of average daily design BOD loading.						
2. CBOD removed - Kg/d						
3. CBOD removed - Kg/hr.						
4. AOR - Kg/hr.						
5. φ- oxygen transfer correction ratio.						
6. SOR - Kg/hr.						

The highest total oxygen transfer rate is used to size the aeration system for the applied CBOD loading.

5.3.3 Odour Control

5.3.3.1 Odour Production

1. Odour Development

Wastewater contains numerous potentially odorous substances, but the predominant group are the reduced sulphur compounds. Of these, hydrogen sulphide is perhaps the most common and the most easily identified. For this reason, odour control measures concentrate on sulphide control.

There are several texts which discuss odour generation, particularly as related to sulphides. The designer is referred to the U.S. EPA Design Manual, "Odour and Corrosion Control in Sanitary Sewage Systems and Treatment Plants" (EPA/625/1-85/018), and the ASCE Manual of Practice No. 69, "Sulfide in Wastewater Collection and Treatment Systems".

2. Odour Measurement and Limits

Odour measurement is largely subjective. The most commonly accepted method of characterization is the 'Odour Unit' (OU). The OU is based on the number of dilutions with clean air required to reach a threshold detection level. OU values are presented as an odour sample's 'Effective Dose' - 50th percentile (ED_{50}), meaning the number of dilutions at which an odour is detected by half the members of an odour panel using a dynamic dilution olfactometer. Thus a sample which requires four dilutions to reach ED_{50} will contain 5 OU (four dilutions plus the original volume).

The designer must determine, in consultation with the Owner and AENV (and often the public), the appropriate odour limits and how they should be applied to the facility. Often a 'fenceline' odour limit is applied, which determines the magnitude of the odours acceptable at the boundary of the facility. The limit will depend on the proximity of residential and commercial development, and other site-specific factors such as the proximity of parks, trails or roads and the sensitivity of the odour problem.

5.3.3.2 Potential Odour Sources

Odour problems tend to develop when dissolved sulphide concentrations exceed 0.5 mg/L, or less if pH is depressed. Sulphide production commences in the collection system, and will continue to occur wherever anaerobic deposits accumulate. The rate of sulphide production and odour generation are both temperature dependent.

Industrial discharges frequently exacerbate odour. Some discharges have high sulphide contents; others may have a low pH or high temperature.

Turbulence promotes sulphide stripping and hence odours. In the collection system, this occurs at drop manholes, sharp bends, forcemain discharge points and any hydraulic structure where turbulence or super-critical flow develops.

Generally, the odour-emission potential at treatment plants decreases at each successive treatment stage. The influent sewer and headworks receive sewage with higher sulphide content and are often turbulent areas. Preliminary treatment processes can generate odours from the screenings and grit handling areas. Aerated grit tanks will also strip sulphides.

Further sulphide generation often occurs as a result of anaerobic action in the sludge blankets accumulating in primary sedimentation tanks. The resultant hydrogen sulphide can be stripped at the effluent weirs due to the turbulence developed there.

Aeration basins do not usually generate high sulphide odours unless overloaded, as sulphides are oxidized within the basin. Attached growth systems are more likely to generate odours, particularly if the growth becomes excessive. Final clarifiers rarely produce significant odours unless there are problems with the sludge or scum handling systems.

Solids handling and treatment processes have significant odour generation potential because of the high sulphide concentrations present in sludge, scum and septage. Aerobic digesters, thickening and dewatering processes and sludge storage lagoons are all potential odour sources.

5.3.3.3 Evaluation of Odour Production Potential

1. Monitoring Protocols

Detailed monitoring exercises should be preceded by a preliminary study to analyze available data and odour complaints. Complaints should be correlated with data on plant operations and wastewater characteristics and meteorological data. The preliminary study may include limited on-site sampling and analyses.

Detailed field monitoring programs should be of sufficient duration to monitor seasonal variations in sulphide generation and Hydrogen Sulphide emissions. Monitoring should also include an hourly sampling and testing regime to identify typical diurnal fluctuations.

Sampling points should be readily identifiable and remain consistent throughout the monitoring program.

i) Liquid Phase Analyses

Routine parameters to be monitored should include total and dissolved sulphides, CBOD or COD, temperature, pH and dissolved oxygen (D.O.). Oxidation-reduction potential (ORP) can also yield useful data.

Additional analyses for TSS and particle size distribution will be required if the results are to be used for predictive modelling of sulphide generation.

ii) Gas Phase Analyses

In-situ gas phase testing can be used to identify a wide range of odorants, including Hydrogen Sulphide, Mercaptans and Dimethyl Disulphide. Continuous monitoring may be necessary in some cases to identify the peak Hydrogen Sulphide gas concentrations which trigger odour complaints. The designer should ensure that the equipment to be used for gas phase testing is suitable for the range of concentrations expected.

iii) Air Sampling

Foul air sampling will be required if it is intended to use a dynamic dilution olfactometer and odour panel to determine OU values. Specialized sampling equipment and sample bags will be required.

iv) Gas Chromatography

GC analyses are useful for identifying total levels of sulphides, and other potential odorants. Analyses can be carried out on liquid or gas phase samples. The collection and preservation of samples shall be in accordance with established procedures for these type of analyses.

2. Interpretation of Results

Data obtained from monitoring programs will provide the designer with useful information concerning the conditions governing sulphide generation and Hydrogen Sulphide release.

Areas of anaerobic activity producing sulphides will be characterized by low D.O. (less than 0.5 mg/L) and negative ORP. Reaction kinetics will be temperature dependent. The rate of sulphide generation will be greater in the presence of higher fractions of soluble BOD.

Hydrogen sulphide emissions will increase at lower pH and higher temperatures.

3. Predictive Modelling

i) Sulphide Generation in Sewers and Forcemains

A number of predictive models have been developed for this purpose. The models have been developed from empirical data and as such are valid only for specific conditions. The designer is cautioned to ensure that the chosen method of analysis is applicable under the conditions in question.

ii) Air Quality Computer Modelling

If designing to specific odour limits at identified receptor points, the designer should consider the use of a computer based atmospheric dispersion model to simulate the behaviour of the odour plume.

5.3.3.4 Odour Control and Abatement Measures

It is generally more reliable and cost effective to treat and remove odorants in the liquid phase rather than collecting and treating foul air. Sulphides develop in the collection system and the designer should therefore consider upstream control measures in conjunction with measures at the wastewater treatment plant. Such measures should include designing to prevent deposition in sewers, minimizing residence time in pump station sumps, and avoiding the use of siphons and long forcemains.

1. Prevention of Sulphide Formation

For new treatment facilities, the design should attempt to eliminate 'dead zones' where solids may accumulate.

Fillets should be incorporated into rectangular channels, conduits and tanks. Inverted siphons should be avoided.

Where self-cleaning velocities cannot be achieved over the full flow range, aeration should be provided to channels and conduits. Excessive aeration should be avoided because of potential odour generation due to increased turbulence. Provide sufficient energy input per unit volume to ensure solids are maintained in suspension.

The designer should also consider improved access provisions to facilitate routine housekeeping and cleaning activities.

2. Chemical Treatment

When dosing chemicals into sewage, side reactions will occur in addition to the desired reaction. In calculating dosing rates, the designer should allow a generous factor of safety to account for these side reactions. Pilot testing is recommended for all chemical dosing systems.

i) Oxidizing Agents

Chlorine (as gas or sodium hypochlorite), potassium permanganate and hydrogen peroxide will oxidize sulphides and inhibit sulphide production. Pure oxygen and air injection have also been used to raise D.O. levels in the sewage.

ii) Precipitants

Iron and zinc salts will precipitate sulphides. Ferrous and Ferric Chloride have been used extensively, in collection systems, forcemains and at wastewater treatment plants. The designer should consider the effect on the solids handling streams at the wastewater treatment plant, in terms of increased sludge production and increased levels of contaminants in the sludge.

iii) pH Control

Intermittent slug dosing with sodium hydroxide will raise the pH, inhibiting sulphide production and preventing hydrogen sulphide off-gassing. This system is effective only in localized areas and should be considered only for specific trouble spots in the collection system.

iv) Electron Acceptors

Electron acceptors are taken up preferentially to the sulphate ion, and thus prevent sulphide formation. Sodium nitrate has been used in lagoons for this purpose. Proprietary nitrate products have also been used in sewers.

3. Control of Mass Transfer

The transfer of sulphides from liquid to gas phase can be reduced by minimizing liquid turbulence and reducing aeration. The designer should consider the following measures to reduce turbulence:

i) Minimize elevation differences where streams converge;

ii) Introduce side streams below the liquid surface;

iii) Use of submerged effluent weirs and downstream flow control in lieu of conventional launders for sedimentation tanks and clarifiers;

iv) Avoid excessive or unnecessary aeration;

v) Avoid the use of screw lift pumps on potentially odorous streams.

4. Foul Air Collection and Treatment

Emission control systems should be considered for all solids handling areas and processes, and for other areas of the facility where preventative measures are insufficient to mitigate odours.

i) Covers

Cover systems should be designed to minimize the number of joints. Seals should be provided at all joints. The designer should consider the corrosive action of sulphides and sulphuric acid when selecting cover materials and concrete coatings. Overhangs, ledges or lips on the underside of covers where condensate may collect should be avoided.

The design of covers should be aimed at minimizing the volume of air requiring treatment.

The designer should consider operational requirements for access and cover removal, and be aware that the installation of covers will create a confined space environment.

ii) Ventilation

Ventilation rates should be based on the more stringent of two requirements: (a) maintain a slight negative pressure in the headspace and thus prevent fugitive odours escaping through joints in the cover system; and (b) limit sulphide concentrations in the air stream to a level which the downstream treatment systems can effectively treat.

iii) Treatment Systems

There are numerous alternative treatment systems available. Selection of the appropriate system should be subject to a life cycle cost-benefit analysis. The systems that may be considered include:

a) chemical scrubbers, packed bed or mist contactor types;

b) activated carbon, with or without chemical impregnation;

c) activated alumina impregnated with potassium permanganate;

d) biofilter, in-vessel or soil/compost types;

e) incineration;

f) dual stage systems comprising one or more of the above.

Design requirements for these systems vary considerably. The designer should consult equipment manufacturers for details.

When assessing bulk chemical storage requirements, the maximum effective storage life of the chemical should be considered. Many chemicals are temperature-sensitive and storage tanks will require special provisions if located outdoors.

Provision will also be required for disposal of waste and side-streams from the treatment systems, which may require further treatment before return to the main liquid stream at the wastewater treatment plant.

An alternative form of biological treatment or pre-treatment involves blowing foul air through aeration basins or trickling filters. If this form of treatment is considered, the designer should be aware of the potential for corrosion in the air blowers and aeration system.

5.3.4 Instrumentation and Controls

5.3.4.1 General

Several factors should be considered when developing a plan for the instrumentation and controls for a wastewater treatment facility. Alberta Environment monitoring requirements vary depending on the type of facility being considered and its location; this will impact on the selection and type of instrumentation being considered. Instrumentation and control requirements will also depend on the size of the plant, and as each treatment process has its own set of conditions to be monitored and controlled there will be different technical requirements to be met. In general, instrumentation and control should provide efficient and safe automatic and manual operation of all plant systems with a minimum of operator effort. Automatic systems should also be provided with manual back-up systems.

The design should have provision for local control systems where parts of the plant may be operated or controlled from a remote location. The local control stations should include provision for preventing the operation of equipment from remote locations.

When making decisions relating to instrumentation and control, the following factors should be considered:

- plant size and complexity
- regulatory requirements
- hours of attended operation
- potential chemical and energy savings
- primary element reliability
- primary element location
- whether controls should be manual **or** automatic
- the data storage and recording requirements and whether data acquisition should be central or distributed.

For effective operation of larger wastewater treatment facilities the following parameters should be measured, some may not be required for smaller facilities:

- flow rate for raw sewage, by-pass flows, final effluent flow
- return Activated Sludge (RAS) flows, Waste Activated Sludge (WAS) flows
- raw and digested sludge flow, digester supernatant flows
- chemical dosage, digester gas production
- hazardous gas monitoring
- anaerobic digester temperature

- dissolved oxygen levels
- sludge blanket levels and sludge concentrations.

5.3.4.2 Types of Instruments

The different types of instruments that may be required to measure the previously mentioned parameters are classified as primary element devices which alter a signal from a physical process to make it suitable for use by a transmitter. These devices are broken down into function groups with a brief description of the process application.

1. Flow Measurement

(i) Magnetic Flow meters (Mag Meters)

Liner and Electrode Materials. The liner for the meter can vary depending on the application being considered. In applications where moderate amounts of abrasion are likely to occur, one of the following materials may be selected; polyurethane, butyl rubber, neoprene or polytetrafluoroethylene. In applications where corrosion is likely to occur, one of the following materials may be selected; ceramic or polytetrafluoroethylene. Stainless steel electrode material should be used for applications where corrosion is not likely to present a problem. Hastelloy electrode material should be used for applications where corrosion is likely to present a problem.

Installation. Installation of magnetic flow meters generally require five straight pipe diameters upstream of the meter and three down stream of the meter free of valves or fittings. Meters may be installed on horizontal, vertical or sloping lines. It is essential to keep the electrodes in the horizontal plane to assure uninterrupted contact with the fluid or slurry being metered. The operating velocity required for these meters will fall into the range or (1-10 m/s) for non-solids bearing liquids and (1.5-7.5 m/s) for solids bearing liquids. When used to meter liquids containing solids, a continuous electrode cleaner or clean out tee should be installed.

Applications. These meters are suitable for Influent Wastewater, Primary Sludge, RAS, WAS, Digested Sludge and Final Effluent These meters should not be used for Digester Gas or liquid streams with a solids content greater than 10% by weight.

(ii) Ultrasonic Flow meters

Flow meter Construction. The flow meter usually consists of an electronics housing, transducers and pipe section. These can in many cases be fitted to existing pipes either by drilling holes for the transducer hardware or by application of external transducers to the outside of the pipe. When installed on existing pipes, the existing pipe material should be checked to assure it will not dampen the sonic signal as this will adversely affect performance.

Installation. The installation of Ultrasonic flow meters generally require ten to twenty straight pipe diameters upstream of the meter and five down stream of the meter free of valves or fittings. Meters can be installed on horizontal, vertical or sloping lines as long as the pipe sections are always full. The operating velocity required for these meters will fall into the range of (1-10 m/s).

Applications. Transmittance style is not recommended for influent wastewater, primary sludge, thickened sludge, nitrification RAS, or nitrification WAS. Reflective style is not recommended for primary effluent, secondary clarifier effluent final effluent or process wash water.

(iii) Turbine Flow meters

Flow meter Construction. The flow meter usually consists of meter body with rotor blades and a magnetic pickup. The pickup is often connected to electronic display units or a totalizer.

Installation. Installation of turbine flow meters generally require a minimum of ten straight and as high as fifty pipe diameters upstream of the meter and five down stream of the meter free of valves or fittings. Meters may be installed on horizontal or vertical pipelines.

Applications. Turbine flow meters are recommended for applications involving natural gas, compressed digester gas.

(iv) Flumes and Weirs (Parshall Flume)

Installation. The flume will be affected by upstream channel arrangement and it is recommended that there be at least ten channel widths upstream. The flume must also be installed carefully to make certain that it is level.

Applications. Flumes and weirs are customarily used to measure flows in open channels. They are recommended for applications involving open channel flow measurement.

2. Suspended Solids Measurement (Turbidity)

Installation. Installation details for turbidity analyzers are unique to each manufacturer; the manufacturer's recommendations should be followed.

Applications. Turbidity analyzers are recommended for applications involving suspended solids concentrations less than 100 mg/L.

3. Suspended Solids Measurement (Optical)

Installation. Installation details for optical analyzers are unique to each manufacturer; the manufacturer's recommendations should be followed.

Applications. Optical analyzers are recommended for applications involving solids concentrations from 20-mg/L to 8%. Examples are, RAS, WAS and mixed liquor.

4. Dissolved Oxygen Measurement (Galvanic)

Installation. Installation details for dissolved oxygen analyzers are usually related to the choice of placement of the analyzer in the process fluid. The analyzers generally require fairly frequent maintenance and this should be considered in determining the location for installation.

Applications. Oxygen analyzers are recommended for applications involving oxygen concentrations from 0-20-mg/L.

5. Level Measurement

(i) Sonic Ultrasonic

Installation. The mounting location of the sensor is determined from restrictions established by the manufacturer. Typically the sensor must be mounted a minimum distance above the high liquid level and should be located away from tank walls or other obstructions that may cause false echoes.

Applications. This type of level element may be used in many level and flow applications; it is not recommended in locations where foam is dense and persistent.

(ii) Float

Installation. Float switches are normally located in a stilling well when turbulence is expected.

Applications. Float switches are commonly used for high and low level alarms and for controlling pump starts and stops.

(iii) Capacitance

Installation. The installation practices can vary and the manufactures recommended installation should be used.

Applications. May be used in applications that require continuous level measurement and also as switches for alarms or start/stop control.

6. Pressure Measurement

(i) Bourdon Tubes

Installation. The installation practice should include the use of block and bleed valves.

Applications. May be used in applications that require pressure indication. Pressure range 0-35000 kPa.

(ii) Bellows

Installation. The installation practice should include the use of block and bleed valves.

Applications. May be used in applications that require pressure indication. Pressure range 0-2000 kPa.

(iii) Diaphragms

Installation. The installation practice should include the use of block and bleed valves. Transmitters should be installed according to manufacturer's recommendations. Temperature extremes should be avoided and location should be as close as possible to the process measure site.

Applications. May be used in applications that require pressure indication or transmitter output. Pressure range 0-3500 kPa.

7. Temperature Measurement

(i) Thermocouples

Installation. The thermocouple should be selected with care to assure that the appropriate device is chosen for the given temperature range. Installation with a thermowell is advised.

Applications. Thermocouples are suitable for most temperature measurement applications.

(ii) Resistance Temperature Detector

Installation. The Resistance detector should be selected with care to assure that the appropriate device is chosen for the given temperature range. Installation with a thermowell is advised.

Applications. Resistance detectors are suitable for temperature measurement applications with ranges of 0-300 C.

(iii) Thermistor

Installation. The Thermistor should be selected to assure that the device is appropriate for the temperature range. Installation with a thermowell is advised.

Applications. Thermistors are suitable for temperature measurement applications with ranges of 0-300 C.

(iv) Thermal Bulb

Installation. No special installation requirements.

Applications. Thermal bulbs are suitable for temperature measurement applications with ranges of 0-500 C.

5.3.4.3 Process Controls

1. Lift Stations

Lift stations require simple and dependable instrumentation and control systems. The parameters that should be monitored are level, flow, pressure, temperatures, hazardous gas levels, as well as status and alarm conditions. The monitoring and control requirements will vary for each individual case based on the size, location, and economic considerations.

(i) Level Control

Lift stations vary in size and storage capacity but generally they require similar controls. The level in the wet well increases to the point where a duty pump will be required to start, a lag and follow pump may be started if the level continues to increase. Pumping continues until a pump stop level is reached at which time the duty pump stops, or a series of stop levels will be reached and the lag and follow pumps stop prior to the duty pump. The pump start/ stop control can be performed using any one of several level elements.

When variable speed pumps are used there are several ways in which the pump can be controlled. These generally centre around control to maintain a level set point in the wet well. This requires a feedback type of control in which the measured variable (level) is compared to a set point value and the final control element is modulated in order to maintain the set point value. Level control of this nature require reliable analog level measurement if it is to function properly. Regardless of the type of level control selected, the system should include a separate low level lockout and high level alarm.

(ii) Flow Monitoring

The flow metering element should be selected carefully to ensure that there are no obstructions where clogging may occur. Provision should be made so that the flow metering element can be bypassed or isolated for routine maintenance activities. The flow metering device should be connected to either the control system or to a recording and totalizing device or both. This provides for a record of flows out of the lift station. It can also be used to help identify possible problems in the discharge piping or force main. [See also Section 5.2.4.2 (5)].

(iii) Pressure Monitoring

Monitoring of the system discharge pressure can be useful in identifying possible problems in the discharge piping or force main and in monitoring pump performance. The pressure metering device should be connected to either the control system or to a recording device or both.

(iv) Pumps and Motors

The following parameters should be monitored:

- pump bearing temperature;
- pump bearing vibration;
- pump speed for variable speed applications;
- pump discharge pressure;
- motor voltage and current;
- motor hours of operation;

- motor bearing temperature; and
- motor windings temperature.

(v) Alarms

Lift stations should be alarmed as outlined in Section 5.2.4.5.

2. Mechanical Bar Screens

Three methods are used to control the operation of mechanical bar screens:

(i) Simple manual start/stop which requires the presence of the operator at the screen in order to start and stop the screen.

(ii) Automatic activation by differential level. This method uses the differential level across the screen to provide the start condition. The screen should run at least one complete screen cycle before stopping. The screen can be called to stop when the differential level is returned to a nil value, the final stop should be controlled using a sensor to determine cycle completion (i.e. limit switch, proximity sensor, timer). In addition, a timer should be provided to initiate a cleaning cycle at regular intervals regardless of actual head loss. When this method is employed, there should be an alarm signal with a head loss set at a point higher than the automatic start of the mechanical bar screen.

(iii) Automatic activation by timer with differential level as emergency start condition. This method uses the differential level across the screen to provide secondary start condition. The screen should run at least one complete screen cycle before stopping. The stop signal should be controlled using a sensor to determine cycle completion (i.e. limit switch, proximity sensor, timer). When this method is employed there should be an alarm signal with a head loss set at a point higher than the automatic start of the mechanical bar screen.

3. Primary Treatment

(i) Raw Sludge Pumping

The raw sludge pumping should be set up to incorporate the following features:

- automatic or manual selection of duty pump;
- on-line sludge density metering for control and monitoring;
- on-line sludge flow monitoring and totalization;
- on-line adjustable sludge density control;
- individually selectable hopper pumping controls where required;
- manual override for automatic controls;
- on-line sludge blanket level monitoring and alarming;
- on-line sludge pump monitoring and control;
- sludge density feedback control for variable speed pumping with manual override;

- on-line sludge pump speed monitoring and control with manual override; and
- on-line monitoring and control of primary tank scraper mechanisms.

(ii) Scum Pumping

The scum pumping should be set up to incorporate the following features:

- automatic or manual selection of duty pump;
- manual override for automatic controls;
- on-line sludge blanket level monitoring and alarming;
- automatic controls consisting of high and low scum tank level for starting and stopping scum pumps;
- high scum tank level alarm;
- on-line scum pump speed monitoring and control with manual override; and
- scum tank flushing system for scum tank cleaning.

4. Secondary Treatment

(i) Dissolved Oxygen (D.O.) Control

Automatic D.O. control systems should be used to control the rate of air supply to aeration tanks. The following methods may be used:

(a) Closed Loop Control (Feedback Control)

Closed loop control consists of on line dissolved oxygen analyzers providing feedback control to an airflow control device. The dissolved oxygen reading is compared to the dissolved oxygen set point. The resultant error signal is used to increase or decrease the rate of airflow to the aeration tanks. Automatic dissolved oxygen control should always be equipped with manual override.

(b) Feed Forward Control

Feed forward control consists of a fixed volume of air being delivered to the aeration tanks for a given flow rate. This system may utilize on line dissolved oxygen analyzers but these are used for monitoring only and do not provide feedback to the airflow control elements.

Process status and alarms should be provided for dissolved oxygen level, blower operating parameters, airflow control elements.

(ii) Return Activated Sludge Control

The Return Activated Sludge pumping should be set up to incorporate the following features:

- automatic or manual selection of duty pump;
- variable speed pumping;
- Return Activated Sludge flow monitoring;
- feedback control to match pumping rates to flow set points;
- individual control of sludge return rate from individual final clarifiers;
- manual override for automatic controls; and
- on-line monitoring of Return Sludge flow rate, pump speed and status.

(iii) Waste Activated Sludge Control

The Waste Activated Sludge pumping should be set up to incorporate the following features:

- automatic or manual selection of duty pumps;
- variable speed pumping;
- Waste Activated Sludge flow monitoring;
- feedback control to match pumping rates to flow set points;
- manual override for automatic controls; and
- on-line monitoring of Waste Sludge flow rate, pump speed and status.

(iv) Chemical Control System

Chemical addition consists of a feeder or chemical metering pump that will dose at a fixed ratio to the influent or effluent flow of the plant, with no analyzer or feedback control. More specific chemical dosing may also be based on such things as Return Sludge flow rate. Chemical dosing requirements will vary widely depending on performance requirements and the specific process being utilized.

(v) Disinfection Control Systems (Ultra Violet)

The disinfection of final plant effluent utilizing Ultra Violet Light consists of a feed forward control system. This consists of a series of lamps and or lamp channels that are turned on based on effluent flow of the plant, UV transmittance analyzers may be utilized for monitoring system performance but are not generally employed in feedback control.

5. Control and Monitoring Systems

Control and monitoring systems can be a conventional system with recorders, indicators, switches, push buttons, indicating lights, control panels, etc. or it can be a computerized control system that utilizes various configurations of hardware and software to provide the control required. Computerized systems can be separated into two groups, PLC (Programmable Logic Controller) Systems and Distributed Control Systems.

(i) Conventional Relay Control Systems

The conventional system is a passive system with limited automatic control, where the operator is responsible for decisions and actions that control the process.

(ii) PLC Control Systems (Programmable Logic Controllers)

The PLC based system is a multipurpose system with extensive scope for modification. The plant status, alarms, motor starters, meters and analyzers are all wired into input/output (I/O) cards located in what are called racks. The racks may be mounted separately or placed in specific plant areas to reduce wiring costs. The I/O racks are associated with controllers that are programmed to perform the required process control functions. Changes can generally be made relatively easily by modification of or addition to the PLC controller programs.

Plant personnel require process information in real-time or in near real-time. The PLC systems accomplish this by means of a Man Machine Interface (MMI). The MMI may be dedicated hardware and software or may come in the form of personal computers utilizing MMI software and connected to the PLC communications system. These systems vary widely in their capabilities and performance. The selection of hardware and software should be done carefully to assure current performance and future supportability and expendability.

5.3.4.4 Design Documents

Complete design documents should be prepared to ensure that construction can be completed correctly and also to properly record the system for future reference. The following are required in the design documents:

- design and construction standards, specifications and installation details
- panel sizing and general arrangements
- control system functional requirements
- control component and instrument data sheets
- operator interface and control hardware and software specifications including input and output (I/O) lists
- control system programming and packaged system configuration standards, structure and scope.

5.3.4.5 Control System Documentation

The following documents should be provided following completion of the control system:

- record drawings to show any changes to the design and including any drawings produced during construction
- annotated listings of control system programs and packaged system configuration
- manufacturer's literature for all control and instrumentation components
- final wiring diagrams complete with wire and terminal coding

- motor control schematics
- instrument loop diagrams
- panel wiring and layout details
- PLC or DCS wiring schematics
- instrument calibration sheets
- operating instructions.

5.3.4.6 Training

Adequate training should be provided to the plant operating and maintenance staff so that the system can be operated to meet the design criteria.

5.3.5 Emergency Facilities and Component Reliability

5.3.5.1 Back-Up Requirements

The components of wastewater treatment plants should be designed in such a way that equipment breakdown and normal maintenance operations can be accommodated without causing serious deterioration of effluent quality.

This does not mean that the tankage capacity has to be doubled. To achieve this, critical treatment processes should be provided in multiple units so that with any unit out of operation, the hydraulic capacity of the remaining units should be sufficient to handle the peak wastewater design flow. There should also be sufficient flexibility in capability of operation so that the normal flow into a unit out of operation can be distributed to all the remaining units.

With some processes such as mechanical screening, the backup can be provided with a less sophisticated unit such as a manually cleaned screen.

Sewage and sludge pumping units should be designed such that with any unit out of service, the remaining pumps operating in parallel should be capable of pumping the peak flows. In certain instances, particularly with sludge pumps, one pump may serve as a back-up for more than one set of pumps, i.e. a raw sludge pump could back-up a sludge transfer pump, etc. Standby capacity requirements for sludge return pumps should be determined on a case-by-case basis.

Aeration systems will require facilities to permit continuous operation, or minimal disruption, in the event of equipment failure. The following factors should be considered when designing the back-up requirements for aeration systems:

- effect on the aeration capacity if a piece of equipment breaks down, or requires maintenance (for instance, the breakdown of one of two blowers will have a greater effect on capacity than the breakdown of one of four mechanical aerators);
- time required to perform the necessary repair and maintenance operations;
- the general availability of spare parts and the time required to obtain delivery and installation.

Generally, considerations such as the above will mean that diffused aeration systems will require a standby blower (maximum air demand should be met with the largest blower out-of-service), but mechanical aeration systems may not require standby units, depending upon the number of duty units, availability of replacement parts, etc.

Chemical feed equipment (e.g. phosphorus removal and disinfection) should be provided in multiple units so that the chemical requirements can be supplied with one unit out of operation.

With sludge digestion facilities, the need for multiple units can often be avoided by providing two-stage digestion along with sufficient flexibility in sludge pumpage and mixing so that one stage can be serviced while the other stage receives the raw sludge pumpage. In smaller plants, multiple primary and secondary digestion units can often be avoided by this method. When such an approach is proposed, the designer should outline the alternate methods of treatment and disposal that could be used during periods of equipment breakdown. With larger treatment plants, the provision of multiple primary and secondary digestion units can usually be economically justified. Single stage digesters will generally not be satisfactory due to the usual need for sludge storage, and effective supernating.

If effluent filtration is employed, Provision of multiple effluent filtration units may be necessary, depending upon the receiving stream sensitivity, type of filtration equipment, and the maintenance requirements of the filter units.

With sludge handling and dewatering equipment, multiple units will generally be required unless satisfactory sludge storage facilities or alternate sludge disposal methods are available for use during periods of equipment repair. The need for full standby units will be unnecessary if the remaining duty units can be operated for additional shifts in the event of equipment breakdown.

5.3.5.2 Wastewater Bypass Facilities

To allow maintenance operations to be carried out, each unit process within the treatment plant should be provided with a bypass facility around the unit.

Where two or more similar treatment units are considered and one unit is out of operation for repairs, the remaining units should be capable of passing the peak wastewater design flow rates or be provided with bypass capacity equal to the excess hydraulic flow of the operating units.

Bypass systems should also be constructed so that each unit process can be separately bypassed.

All flows bypassing secondary and/or tertiary treatment processes should be measured.

5.3.5.3 Standby Power

1. General

Plants should be provided with an alternate source of electric power or pumping capability to allow continuity of operation during power failures, except as noted below. Methods of providing alternate sources include:

(a) the connection of at least two independent power sources such as substations. A power line from each substation should be installed;

(b) portable or in-place internal combustion engine equipment which will generate electrical or mechanical energy; and

(c) portable pumping equipment when only emergency pumping is required.

2. Power for Aeration

Standby generating capacity normally is not required for aeration equipment used in the activated sludge process. In cases where a history of long-term (four hours or more) power outages have occurred, auxiliary power for minimum aeration of the activated sludge may be provided.

3. Power for Disinfection

When receiving water stream is environmentally sensitive, continuous disinfection should be provided during all power outages.

5.4 Residual/Biosolids (Sludge) Processing

5.4.1 General

The treatment, handling and disposal of wastewater sludges should be integrated with the planning and design of all wastewater treatment plants. The purpose of sludge processing is to reduce and stabilize biodegradable organic matter so that handling and disposal may be done in an environmentally acceptable manner. Techniques for processing and disposing of sludges will depend on characteristics of the wastewaters and sludges, the wastewater treatment process, and the size and location of the wastewater treatment facility.

Facilities for processing sludge should be provided at all mechanical wastewater treatment plants. Handling equipment should be capable of processing sludge to a form suitable for ultimate disposal. If ultimate disposal method is not suitable year round, provision must be made for sludge storage during the period disposal is not possible.

The selection of sludge handling unit processes should be based upon at least the following considerations:

1. local land use;
2. system energy requirements;
3. cost effectiveness of sludge thickening and dewatering;
4. sludge digestion or stabilization requirements;
5. sludge storage requirements; and
6. methods of ultimate disposal.

5.4.2 Biosolids (Sludge) Handling and Treatment

5.4.2.1 Digestion

Sludge stabilization is generally achieved by digestion. Two types of digestion systems are used - anaerobic and aerobic.

Anaerobic digestion is the most commonly used system for the digestion of primary and mixtures of primary and waste activated sludges. Aerobic digestion, because of the relatively large energy requirements, is not recommended for use, particularly at large wastewater treatment facilities.

1. Anaerobic Digestion

Anaerobic sludge digestion is used to reduce and stabilize the biodegradable organic matter to improve the dewatering characteristics of sludge and to reduce pathogenic organisms. Bulky, odorous raw sludges are converted to a relatively inert material that can be readily dewatered in the absence of offensive odours. Oxygen is excluded from the anaerobic digestion process, and most pathogenic organisms are destroyed by properly designed digester tanks operating at temperatures around 35°C over a period of about 15 to 30 days. Temperature, pH, mixing, and retention time are the critical design factors which should be considered.

Anaerobic digesters should meet the requirements of the latest edition of "CAN/CGA-B105, Code for Digester Gas and Landfill Gas Stations. Digestion systems should also be designed with features and in accordance with design parameters, as follows:

- *Number of Stages*

  Two (primary and secondary).

- *Number of Digesters in Each Stage*

  One adequate in small plants provided that flexibility is given to allow either stage to receive raw sludge in emergencies; number of digesters in each stage of large plants will be dictated by economics.

- *Hydraulic Retention Time in Primary Digester*

  Minimum 15 days (sludge retention time requirements of slowest methane producers is approximately 10 days).

- *Mixing*

  For digestion systems utilizing two stages, the first stage (primary) should be completely mixed (via digester gas - compressor power requirements 5 to 8 W/m^3 or mechanical means - 6.6 W/m^3). The second stage (secondary) is to be designed for sludge storage, concentration, and gas collection and should not be credited in the calculations for volumes required for sludge digestion.

- *Volatile Solids Loading*

  0.5 - 1.5 kg/m^3/d.

- *Completely Mixed Systems*

  For digestion systems providing for intimate and effective mixing of the digester contents, the system may be loaded with volatile solids up to 1.3 kg/m^3 of volume per day in the active digestion units.

- *Moderately Mixed Systems*

  For digestion systems where mixing is accomplished only by circulating sludge through an external heat exchanger, the system may be loaded with volatile solids up to 0.65 kg/m^3 of volume per day in the active digestion units. This loading may be modified upward or downward depending upon the degree of mixing provided.

- *Heating*

Heating must be provided for the primary digester so that a temperature of 35°C can be maintained. External heat exchanger systems are preferred. Heating should be via a dual-fuel boiler system using digester gas and natural gas, or oil.

- *Digester Covers*

Digester covers may be fixed or floating type, sized to provide gas storage volume. Insulated pressure and vacuum relief values and flame traps should be provided. Access manholes and sampling wells should also be provided in the digesters covers. The underside of the roof must be protected from corrosion. Coatings should be compatible with the roofing material and the environment in the digester to ensure a good bond.

Steel, concrete or fibreglass covers may be used.

- *Secondary Digester Sizing*

The secondary digester should be sized to permit solids settling for decanting and solids thickening operations, and in conjunction with possible off-site facilities, to provide the necessary digested sludge storage. Off-site storage in sludge lagoons, sludge storage tanks, or other facilities, may be used to supplement the storage capacity of the secondary digester.

- *Sludge Piping*

Maximum flexibility should be provided in terms of sludge transfer from primary and secondary treatment units to the digesters, between the primary and secondary digesters, and from the digesters to subsequent sludge handling operations; minimum diameter of sludge pipes should be 100 mm; provision should be made for flushing and cleaning sludge piping; sampling points should be provided on all sludge lines; main sludge transfer lines should be from the bottom of the primary digester to the mid-point of the secondary digester.

- *Supernatant Piping*

Supernatant should be returned to the treatment plant with flexible points of return to the grit removal facilities, upstream of the primary settling tanks, or to the aeration tank; multiple draw-off points or adjustable supernatant draw-offs, and sampling points should be provided; both primary and secondary digesters should be equipped with supernatant piping so that during emergencies the primary can be operated as a single stage process; additional $CBOD_5$ load caused by supernatant return should be considered in aeration system design.

- *Overflows*

Each digester should be equipped with an emergency overflow system.

- *Waste Gas*

i) Location

Waste gas burners shall be readily accessible and should be located at least 15 m away from any plant structure. Waste gas burners shall be of sufficient height and so located to prevent injury to personnel due to wind or downdraft conditions.

ii) Pilot Light

All waste gas burners shall be equipped with automatic ignition such as a pilot light or a device using a photoelectric cell sensor. Consideration should be given to the use of natural or propane gas to insure reliability of the pilot.

iii) Gas Piping Slope

Gas piping shall be sloped at a minimum of 2 percent up to the waste gas burner with a condensate trap provided in a location not subject to freezing.

2. Aerobic Digestion

Aerobic digesters treating waste activated sludge should be designed in accordance with the following criteria. If primary sludge is to be included, minimum sludge age and air requirements may have to be increased.

- *Number of Stages*

Two.

- *Number of Tanks in Each Stage*

Generally one.

- *Loading*

1.5 kg/m^3.d volatile suspended solids based upon first stage volume only.

- *Sizing*

Designed to achieve a minimum sludge age of 45 days, including both stages and sludge age of waste activated sludge; if a total of 45 days sludge age is all that is provided, it is suggested that 2/3 of the total digester volume be in the first stage and 1/3 be in the second stage; if major additional storage volumes are required, separate on-site or off-site sludge storage facilities should be considered to avoid the power requirements associated with aerating greatly oversized aerobic digesters.

- *Air and Mixing Requirements*

  Aeration rate will depend upon the oxygen uptake rate at the maximum solids content experienced; as a guideline, 0.85 L/m^3.s (litres of air per cubic metre of aeration tank per second) should be provided for diffused aeration systems; a minimum bottom velocity of 0.25 m/s should be maintained while aerating; mechanical surface aeration systems are not recommended due to increased heat loss causing icing problems.

- *Tank Design*

  Generally open; tankage should be of common wall construction or earthern-bermed to minimize heat loss; tank depths 3.6-4.6 m; tanks and piping should be designed to permit sludge addition, sludge withdrawal, and supernatant decanting from various depths to, or from both the primary and secondary digester.

5.4.2.2 Conditioning

Sludge dewatering, and to a lesser extent sludge thickening operations, are highly dependent upon sludge conditioning for their successful operation. Sludge conditioning not only affects the solids concentration of the thickened or dewatered sludge, but also affects the solids capture efficiency of the process.

There are two sludge conditioning approaches that can be used. Sludge can be conditioned by physical methods, such as heat treatment, or by chemical methods, involving the addition of either organic or inorganic chemicals.

The method selected will not only differ in its effect on the thickening or dewatering process, but will have different effects on subsequent sludges handling operations and on the sewage treatment process itself.

1. **Physical Methods**

   Heat conditioning of sludge consists of subjecting the sludge to high levels of heat and pressure. With this process, the sludge is treated at temperatures of 175 to 204°C, pressure of 1700 to 2800 kPa and for detention times of 15 to 40 minutes. The high temperatures cause hydrolysis of the water-solids matrix and breaking down of the biological cells. The hydrolysis of the water matrix destroys the gelatinous components of the organic solids and thereby improves the water-solids separation characteristics.

   Although the heat conditioning system has been proven to be an effective sludge conditioning technique for subsequent dewatering operations, the process results in a significant organic loading to the aeration tanks of the sewage treatment plant, if the supernatant is returned to the aeration system, due to the solubilization of organic matter during the sludge hydrolysis. This liquor can represent 25 to 50 percent of the total loading on the aeration tanks and allowances must be made in the treatment plant design to accommodate this loading increase.

   Heat conditioning results in the production of extremely corrosive liquids requiring the use of corrosion-resistant materials such as stainless steel. Scale formation in the heat exchangers, pipes and reactor is a common problem.

The design requirements for a heat conditioning system should be determined by either batch or small-scale continuous pilot plants. Through such methods, the necessary level of hydrolysis to produce the desired reduction in the specific resistance of the sludge, and the liquor characteristics can be determined. Tests can also be made at different temperatures and retention times to determine the most effective full-scale operating conditions.

Freezing of sludges has been used successfully for water treatment plant sludges, but not common as a conditioning method for sewage sludges.

2. Chemical Methods

Chemical conditioning methods involve the use of organic or inorganic flocculants to promote the formation of a porous, free draining cake structure. Chemical conditioning for thickening operations attempts to promote more rapid phase separation, higher solids concentration and a greater degree of solids capture. With dewatering operations, chemical conditioning is used in an attempt to enhance the degree of solids capture by destabilization and agglomeration of fine particles. This promotes the formation of a cake which then becomes the true filter media in the dewatering process.

With most thickening operations and with belt filter press dewatering operations the most commonly used chemicals are high molecular weight polymers. The selection of the most suitable chemical(s) and the dosage requirements for sludge conditioning can be best determined initially by bench and pilot testing.

Laboratory testing should, however, be used to narrow down the selection process and to arrive at approximate dosage requirements. Generally, laboratory testing will yield dosage requirements within 15 percent of full-scale needs.

5.4.2.3 Thickening

Sludge thickening can be employed in the following locations in a wastewater treatment plant:

- prior to digestion for raw primary, excess activated sludge or mixed sludges;
- prior to dewatering facilities;
- following digestion for sludges or supernatant;
- following dewatering facilities for concentration of filtrate, decant, centrate, etc.

The commonly employed methods of sludge thickening are gravity and air flotation. Their suitability for the various types of sludge are shown in Table 5.10. Centrifuges, gravity belt thickeners and rotating drum thickeners are also used for sludge thickening. All thickening devices are adversely affected by high sludge volume indexes (SVIs) and benefited by low SVIs in the influent activated sludges. The ranges of thickened sludge concentrations given in Table 5.9 assume an SVI of approximately 100.

Wherever thickening devices are being installed special consideration should be given to the need for sludge pre-treatment in the form of sludge grinding to avoid plugging pumps, lines, and thickening equipment. Also, where thickeners are to be housed, adequate ventilation should be provided.

1. Gravity

Gravity thickening is principally used for primary sludge, and mixtures of primary and waste activated sludges, with little use for waste activated sludges alone.

Gravity thickeners should be designed in accordance with the following parameters:

- *Tank Shape*

  Circular.

- *Tank Depth*

  3 to 3.7 m.

- *Tank Diameter*

  Up to 21 - 24 m.

- *Floor Slope*

  Acceptable range 2:12 to 3:12.

- *Solids Loading*

  Primary sludges 96 to 120 kg/m^2.d; waste activated 12 to 36 kg/m^2/.d; combination of primary and waste activated based on weighted average of above loading rates.

- *Overflow Rate*

  0.19 to 0.38 L/m^2.s.

- *Chemical Conditioning*

  Provision should be made for the addition of conditioning chemicals into the sludge influent lines.

- *Sludge Volume Ratio*

  Volume of sludge blanket divided by volume of sludge withdrawn daily should be 0.5 to 2 days.

TABLE 5.10
SLUDGE THICKENING METHODS AND PERFORMANCE WITH VARIOUS SLUDGE TYPES

Thickening Method	Sludge Type	Performance Expected
GRAVITY	Raw Primary	Good, 8 to 10% Solids
	Raw Primary and Waste Activated	Poor, 5 to 8% Solids
	Waste Activated	Very Poor, 2 to 3% Solids (Better results reported for oxygen excess activated sludge)
	Digested Primary	Very Good, 8 to 14% Solids
DISSOLVED AIR FLOTATION	Waste Activated (Not generally used for other sludge types)	Good, 4 to 6% Solids and _ 95% Solids Capture

2. Air Flotation

Unlike heavy sludges, such as primary and mixtures of primary and excess activated sludges, which are generally most effectively thickened in gravity thickeners, light excess activated sludges can be successfully thickened by flotation.

Flotation operations cannot be designed on the basis of purely mathematical formulations or by the use of generalized design parameters and some bench-scale and/or pilot-scale testing will be necessary. The following design parameters are given only as a guide to indicate the normal range of values experienced in full-scale operation:

- *Tank Dimensions*

  Vary with suppliers.

- *Air Buoyancy Systems*

  Vary with suppliers.

- *Air to Solids Weight Ratio*

  0.02 to 0.05.

- *Recycle Ratios*

  Vary with suppliers (0 to 500%).

- *Solids Loadings (with waste activated sludge to achieve 5% float solids)*

  48 $kg/m^2.d$ (without flocculating chemicals); up to 240 $kg/m^2.d$ (with flocculating chemicals).

- *Chemical Conditioning*

  Feed chemical to mixing zone of sludge and recycled flow.

- *Hydraulic Feed*

  Up to 1.74 L/m^2/.s (based on total flow including recycle, when polymers used); without chemicals, lower rate must be used; feed rate should be continuous.

- *Detention Time*

  Not critical provided particle rise rate is sufficient and horizontal velocity in the unit does not produce scouring of the sludge blanket.

5.4.2.4 Dewatering

Sludge dewatering will generally be required prior to ultimate disposal of sludges, other than for land application. Since the processes differ significantly in their ability to reduce the water content of sludges, the ultimate sludge disposal method will generally have a major influence on the dewatering method most suitable for a particular wastewater treatment plant. Also of influence will be the characteristics of the sludge requiring dewatering, that is, whether the sludge is raw or digested, whether the sludge contains waste activated sludge, or whether the sludge has been previously thickened.

Table 5.11 gives the solids capture, solids concentrations normally achieved, energy requirements and suitable ultimate disposal options for various dewatering methods. The solids concentrations shown in Table 5.11 assume that the sludges have been properly conditioned.

In Alberta, the requirement for landfilling municipal sludges is dictated by Public Health Regulations. In general, the solids concentration for sludges which are to be landfilled at sanitary landfill sites will be influenced by the quantities of sludge to be disposed of in relation to the quantities of municipal refuse, characteristics of the site itself, and the expected effects of the liquid addition to the site. With small quantities of sludge for co-disposal landfilling with garbage, liquid sludge at solids concentrations as low as three (3) percent may be acceptable. For sludge only landfill operations, a minimum of 15 percent solids concentration is generally required to support cover material.

If sludge is to be disposed of in sludge lagoons, dewatering may not be necessary unless it is justifiable for economic reasons relating to haulage costs.

In Alberta, the most prevalent method of sludge disposal is by land application on agricultural lands. Due to the fact that the ammonium nitrogen content of sludges is largely associated with the liquid fraction of sewage sludges and the acceptability of sludges for spreading on agricultural land relates to minimum ratios of nitrogen to heavy metal concentrations, dewatered sludges will generally be less desirable for final spreading on agricultural lands than liquid sludges. To enable sludges to be handled and spread as liquids, the upper limit for solids content will generally be in the order of 12 percent. This would leave only thickened sludges acceptable for spreading on agricultural land by liquid spreading techniques, and sludge dewatering will not be necessary. For this reason, no design guidelines have been included in this document for sludge dewatering facilities. However, should it become necessary to dewater sludges, the proponent should consult with AENV before proceeding with the design of the facility.

TABLE 5.11
SLUDGE DEWATERING METHODS AND PERFORMANCE WITH VARIOUS SLUDGE TYPES

Dewatering Method	**Solids Capture (%)**	**Solids Concentrations Normally Achieved**	**Median Energy Required (m^3/dry tonne)**	**Suitable Ultimate Disposal Methods**	
				Landfill	**Agricultural Utilization**
Filter Press	90 to 95	Raw Primary + WAS- 30 to 50%) Digested Primary + WAS- 35 to 50%) WAS- (25 to 50%)	360	Yes Yes Yes	No No No
Centrifuge (Solid Bowl)	95 to 99	Raw or Digested Primary + WAS - (15 to 25%) WAS - (12 to 15%)	360	Yes Yes	No No
Belt Filter	85 to 95	Raw or Digested Primary + WAS - (14 to 25%) WAS - (10 to 15%)	130	Yes Yes	No No

6.0 Stormwater Management Guidelines

6.1 General

This section provides a brief summary of the design standards and guidelines for storm drainage systems in Alberta. Detailed stormwater management standards and guidelines are described in the AENV publication entitled, Stormwater Management Guidelines for the Province of Alberta.

6.2 Stormwater Collection

6.2.1 Dual Drainage Concept

Dual drainage concept (minor and major systems) should be followed in the design of the collection systems. The minor system (underground pipe systems, roof leaders, gutters, lot drainage, etc.) provides a basic level of service by conveying flows during minor storm events; the major system (lot drainage, roads and gutters, storage facilities, etc.) conveys runoff from the extreme events in excess of the minor system capacity.

There is always a major system, whether or not one is planned. Failure to plan for a major system often results in unnecessary flood damage.

6.2.1.1 Design Capacity

The establishment of capacity criteria for the minor system is largely a trade-off between cost and convenience in terms of level of service. For larger municipalities, the minor system should be designed to carry the peak flow resulting from a one in 5-year rainfall event; for several communities faced with limited financial reserves, the use of the 2-year event may be practical.

For the major system, the design should be based on a one in 100-year rainfall event.

6.2.2 Storm Sewers

Storm sewers shall be designed as a separate sewer system. Effluent from sanitary sewers or any potentially contaminated drainage from industrial, agricultural, or commercial operations shall not be discharged to storm sewers.

Contaminated drainage means, the introduction of any foreign, undesirable physical, chemical or biological substance into the environment which results or is likely to result in deleterious effects.

6.2.2.1 Sewer Hydraulics

Storm sewer pipe shall be designed to convey the design flow when flowing full with the hydraulic grade-line at the pipe crown. Crown elevations should match at manhole junctions.

6.2.2.2 Flow Velocities and Minimum Slope

Storm sewer flow velocities shall not be less than 0.60 m/s when flowing full. Refer to Table 6.1 for minimum slopes of gravity storm sewers.

If sewer flow velocities exceed 3 m/s, special consideration shall be given to prevent scouring.

TABLE 6.1
MINIMUM DESIGN SLOPES FOR STORM SEWERS

Sewer Diameter (mm)	Minimum Design Slope (m/100m)
300	.194
375	.145
450	.114
525	.092
600	.077
675	.065
750	.057
900	.045
1050	.036
1200	.031
1350	.027
1500	.023
1650	.020
1800	.018
1950	.016
2100	.015
2250	.013
2400	.012
2550	.011
2820	.010

Note: Design slopes based on a minimum velocity of 0.60 m/s for pipe flowing at least half full

6.2.2.3 Pipe Size

The minimum diameter for storm sewers shall be 300 mm. The minimum diameter for catch basin leads shall be 250 mm.

6.2.2.4 Pipe Material

The selection of pipe material, pipe classes and bedding types should be based on loading conditions. The designer should be particularly careful to specify sulphate resistant concrete pipe in areas of sulphate soil.

Storm sewer pipe shall have been manufactured in conformity with the latest standards by the American Society for Testing Materials (ASTM) or the Canadian Standards Association (CSA).

6.2.2.5 Pipe Cover

The minimum depth of cover to pipe crown shall be 1.2 m.

6.2.2.6 Curved Sewers

Curved sewers shall match the roadway curvature by means of deflection at the joints only. Joint deflections shall not exceed the manufacturer's specified allowable deflection. Consideration should also be given to increasing the grade of curved sewers to offset increased head loss.

6.2.2.7 Change in Flow Direction

For storm sewer pipes greater than 600 mm in diameter, changes in flow direction at manholes should not exceed 45^{o}. This limitation may be exceeded if care is taken to design a proper transition manhole.

6.2.2.8 Extraneous Flows

Roof leaders shall not be connected to storm sewers in residential areas, but shall discharge to grassed or pervious areas. Roof leaders from multi-family, commercial, and industrial sites and foundation drains may be connected to storm sewers at the discretion of the Local Authority.

6.2.2.9 Sewer Maintenance

Control should be provided to minimize sediment discharge to storm sewers. This control may be in the form of properly graded and surfaced streets and lanes, landscaping, catch basin sumps, or sediment control structures at pond and lake inlets.

6.2.3 Storm Manholes

The design of storm manholes should conform in all respects to Section 5.2.2 pertaining to the design of sanitary sewer manholes, with the following exception:

- For storm sewers, 1.0 m in diameter or larger, a bend may be installed instead of a manhole at all changes in grade or alignment.

6.2.4 Catch Basins and Gutters

6.2.4.1 Collection of Surface Runoff

Surface water should not be permitted to run a distance greater than 300 m along roadways without interception by the first catch basin. From this first point of interception, surface runoff should not run a distance greater than 120 m between catch basins.

6.2.4.2 Catch Basin Capacity

The inlet capacity of each catch basin should be sufficient to receive the calculated surface stormwater flow at that location. The minimum inside diameter of catch basin leads shall be 250 mm.

6.2.4.3 Catch Basin Construction

All catch basin bodies shall be of either 600 mm or 900 mm pre-cast concrete sections. Where a sump cleaning maintenance program is in effect, the body shall be constructed so as to provide a 600 mm sump to trap silt and gravel.

6.2.4.4 Gutters

The minimum grade of gutters used to intercept stormwater runoff should be 0.40%. Gutters of less than 20 m in length or curved gutters of short radius should have a minimum grade of 0.60%.

6.2.5 Stormwater Pumping Stations

Being that stormwater pumping is an uncommon practice, there are no specific criteria in these standards with respect to the design and operation of stormwater pumping stations. The proponent of a pumping station should contact the Regional Director of Alberta Environment before commencing with detailed design.

6.3 Stormwater Best Management Practices (BMPs)

6.3.1 Introduction

6.3.1.1 General

Stormwater Best Management Practices (BMPs) are methods of managing stormwater drainage for adequate conveyance and flood control and are economically acceptable to the community. BMPs are stormwater management methods that retain as much of the "natural" runoff characteristics and infiltration components of the undeveloped system as possible and reduce or prevent water quality degradation.

Stormwater BMPs that may be considered for stormwater quantity and quality controls are discussed in the following order:

- source control BMPs
- lot-level BMPs
- conveyance system BMPs
- end-of-pipe BMPs.

6.3.1.2 Design Criteria for Stormwater Quality Control

It is considered that storing the volume of runoff from a 25-mm storm over the contributing area is appropriate for Alberta for stormwater quality control using detention devices such as dry ponds, wet ponds, and constructed wetlands. A detention time of 24 hours should also be used for detention facilities. The runoff from a 12-mm storm over the contributing area is considered appropriate for infiltration BMPs.

6.3.2 Source Control BMPs

Removal of stormwater contaminants at their source may, in some instances, be a practical solution to the mitigation of pollutant impacts. There are three main pollutant removal activities that are normally practiced by a municipality for source control including street sweeping, catch basin cleaning, and animal litter removal through enforcement of bylaws.

6.3.3 Lot-Level BMPs

Stormwater lot-level controls are practices that reduce runoff volumes and/or treat stormwater before it reaches a subdivision/development conveyance system. This type of controls can be readily incorporated into the design of future developments. With all development, the applicability of stormwater lot-level controls should be investigated before conveyance and end-of-pipe systems are examined.

Traditional lot-level controls aimed at stormwater quantity management and the reduction of peak runoff rates include:

- restricting numbers of roof drains to provide rooftop detention of stormwater,
- installing catch basin restrictors or orifices in the storm sewer to promote parking lot detention,
- over sizing storm sewers and installing orifices in the sewer to create pipe storage,
- installing catch basin restrictors in rear yard catch basins to create rear yard storage.

The above-noted lot-level measures are primarily designed to reduce runoff peaks. Other stormwater management criteria, such as the preservation of water quality, protection from erosion, and the maintenance of base flow are not adequately addressed through these techniques. Lot-level controls that help preserve the natural hydrologic regime include:

- reduced lot grading,
- directing roof leaders to rear yard ponding or soak away pits,
- sump pumping foundation drains to rear yard ponding areas.

6.3.3.1 Reduced Lot Grading

1. Purpose

The purpose of reducing lot grades is to reduce the volume of runoff from developed lots by increasing the travel time of runoff, and increasing the availability and opportunity for depression storage and infiltration. A significant reduction in lot-level runoff volumes would also affect the other minor stormwater system components and the major system components by reducing the conveyance and treatment requirements.

2. Description

Typical development standards require a minimum lot grade of two percent to drain stormwater away from buildings. In flat areas, a reduction to minimum lot grades should be evaluated. In hilly areas, alterations to natural topography should be minimized. To avoid foundation drainage problems, the grading within 2 to 4 m of buildings should be maintained at two percent or higher. Areas outside of this envelope should be graded at less than two percent.

Reduced lot grading BMPs promote depression storage and natural infiltration and reduce risks associated with flooding and erosion. The maintenance of natural infiltration could have positive impacts on base flow depending on local evapotranspiration rates.

3. Applicability

Reduced lot grades can be recommended as a lot-level stormwater BMP for any new developments and in re-grading or re-landscaping of existing lots in established developments.

4. Effectiveness

Very little information is available in regard to the impact that reductions in lot grades may have on the overall runoff volumes from a developed area. It has been recommended that reductions in lot grading may increase the pervious depression storage by as much as 1.5 mm for a 0.5 percent to 2.0 percent change in grade. Reduction of on-lot runoff will also reduce downstream erosion potential.

5. Water Quantity

Reduced lot gradings limit the volumes of runoff normally directed toward minor drainage systems. On-lot drainage rates are also reduced. This will reduce the requirements for end-of-pipe detention storage. Effective on-lot drainage reductions on a subdivision basis will lower and flatten the receiving water inflow hydrograph.

Increased infiltration of stormwater also provides recharge to the local groundwater that may, in turn, discharge to local streams thus enhancing base flows.

6. Water Quality

Reduced lot gradings limit the volumes of runoff from smaller storm events that are normally the major contributor of receiving water contaminants. The effectiveness of reduced lot grades in limiting contaminant runoff is also dependent on land use.

7. Design Considerations

Design guidelines for on-lot grade reductions are shown in Figure 6-1. Grades within 4 m of structures should be maintained at two percent. Grades beyond 4 m of structures should be reduced to 0.5 percent. Consideration should also be given to tilling soils in flatter grade areas to a depth of 30 mm prior to seeding or sodding to reduce soil compaction and increase infiltration potential.

6.3.3.2 Surface Ponding and Rooftop Storage

1. **Purpose**

Roof leaders that discharge to surface ponding areas reduce the potential for downstream flooding and erosion and help maintain pre-development end-of-pipe discharge rates. The same benefits can result from the use of rooftop storage, which are likely suitable for commercial, industrial, and institutional buildings.

2. **Description**

Roof leaders are directed toward rear lot depressions that allow stormwater to infiltrate or evaporate. For rooftop storage roof, drains on flat roofs are raised to allow ponding on the rooftop.

3. **Applicability**

Surface ponding areas can be recommended as a lot-level stormwater BMP for any new developments and in re-grading or re-landscaping of existing lots in established developments. Surface ponding may also be used for parking lots or park areas. Rooftop storage can be recommended for industrial, commercial, or institutional buildings with flat roofs.

4. **Effectiveness**

Rear lot ponding of stormwater or rooftop storage effectively limits runoff by a volume equal to the amount of impervious depression storage provided.

5. **Water Quantity**

Rear lot ponding and rooftop storage limit the volumes of runoff normally directed toward minor drainage systems. On-lot drainage rates are also reduced. This will reduce the requirements for end-of-pipe detention storage. Effective on-lot drainage reductions on a subdivision basis will lower and flatten the receiving water inflow hydrograph.

Increased infiltration of stormwater from rear lot ponds also provides recharge to the local groundwater which may in turn discharge to local streams thus enhancing base flows.

6. **Water Quality**

Rear lot ponding and rooftop storage limit the volumes of runoff from smaller storm events that are normally the major contributor of receiving water contaminants.

7. **Design Considerations**

Design guidelines for rear lot ponds are shown in Figure 6-2. Maximum depths should be maintained at 100 mm. Flow paths should be provided to direct overland flow to the pond. To maintain the pond, catch basins can be elevated to the required height or grassed swales can be created. More complex designs may incorporate an infiltration trench beneath the ponded area to enhance infiltration. The pond should be sized to accommodate a minimum of 5 mm and a maximum of 20 mm of rainfall covering the roof area. Rooftop ponding can be accomplished by raising roof hoppers to create a maximum ponding depth of 10 mm.

6.3.3.3 On-lot Infiltration Systems

1. Purpose

On-lot infiltration systems are used for detention of stormwater from relatively small catchment areas. Infiltration systems may be used in areas without adequate minor system conveyance. They also provide enhancement to water quality and reductions in overland flow.

2. Description

Infiltration systems may be simply designed pits with a filter liner and rock drain material or more complex systems with catch basin sumps and inspection wells. Stormwater flow from roof drains is directed to the infiltration system.

3. Applicability

Infiltration systems are recommended for relatively small detention volumes. If larger detention volumes are required a series of infiltration basins may be employed. Infiltration basins should not be built under parking lots or other multi-use areas, if the groundwater table is within 0.6 m of the infiltrating surface, if bedrock is located within 1.2 m of the infiltration surface, if the infiltrating surface is located on top of fill material and if the underlying soils have a fully saturated percolation rate of less than 1.3 mm.

4. Effectiveness

Infiltration systems have a number of advantages over rear yard ponding including increased groundwater recharge and less inconvenience to homeowners. Infiltration systems may have increased maintenance requirements over ponds and a more uncertain operating life. On-lot infiltration systems accept only roof runoff and are therefore subjected to minimal levels of suspended solids.

5. Water Quantity

On-lot infiltration systems limit the volumes of runoff normally directed toward minor drainage systems. On-lot drainage rates are also reduced. This will reduce the requirements for end-of-pipe detention storage. Effective on-lot drainage reductions on a subdivision basis will lower and flatten the receiving water inflow hydrograph.

Increased infiltration of stormwater from rear lot ponds also provides recharge to the local groundwater which may in turn discharge to local streams thus enhancing base flows.

6. Water Quality

On-lot infiltration systems limit the volumes of runoff from smaller storm events that are normally the major contributor of receiving water contaminants.

7. Design Considerations

Figures 6-3 and 6-4 illustrate two different applications of infiltration systems. The total void volume should be calculated from the storage required for the 2-year design storm which is calculated from the effective porosity of the infiltration fill material. The infiltration surface area required (bottom surface area) to drain the system within 48 hours is calculated from the 24-hour sustained percolation rate. An overland flow path should be provided for overflow volumes during saturated or frozen conditions. A pre-treatment filter (Figure 6-3) or sump (Figure 6-4) should be provided to limit solids input into the system. Design void space volumes are calculated from the volume of water required to fill a known volume of drain rock. A suitable quality filter fabric or geotextile must also be incorporated into the design.

In locating infiltration systems, consideration should be given to proximity to septic fields.

6.3.3.4 Sump Pumping of Foundation Drains

1. Purpose

Many current development standards allow foundation drains to be directly connected to the storm sewer. By pumping foundation drainage to surface or subsurface ponding/soak away areas, infiltration, flooding, and erosion water management concerns may be reduced.

2. Description

Foundation drainage is sometimes pumped to the storm sewer network, to a suitable infiltration system, or to the surface where it is conveyed to a catch basin and then to a storm sewer.

3. Applicability

Sump pumps are not feasible in areas where the seasonal high groundwater table is within 1 m of the foundation drain. Sump pumps are not feasible in areas where bedrock is within 1 m of the foundation drain. Application under these conditions may cause excess pumping. Under other conditions where infiltration systems are appropriate or where overland flow paths are available, sump pumps can be recommended to discharge to either the infiltration system or to the surface.

4. Effectiveness

Foundation drainage is normally relatively clean water and is well suited to the optimal operation of infiltration systems or overland flow to rear yard ponds.

5. Water Quantity

The impact of foundation drain discharge on downstream stormwater management facilities is dependent on the original discharge location. If foundation drainage was originally discharged to the storm sewer network or to the sanitary sewer, there will be some reduction in stormwater flow in the sewer. There will also be additional groundwater recharge and potentially base flow augmentation in the local receiving stream if foundation drainage was originally discharged to either the storm sewer or sanitary sewer networks.

6. Water Quality

Foundation drainage is relatively clean water and if flow is removed from either the storm sewer network or the sanitary sewer network there is likely to be some impact on the dilution of contaminants provided by the foundation drainage.

7. Design Considerations

Sump pump drainage to an infiltration system is illustrated in Figure 6-5. The location of the infiltration system should conform to infiltration design considerations. Yard grades should conform to design considerations for infiltration ponds. Sump pump discharges should be located at least 2.0 m away from foundations and be discharged to rear yards away from sidewalks to prevent icing conditions during winter months. Discharges should also be located at least 0.5 m above ground to prevent blockage from ice and snow during the winter.

6.3.4 Stormwater Conveyance System BMPs

Stormwater conveyance systems transport drainage from developed areas through sewer or grassed swale systems. Stormwater conveyance controls are applied as part of the stormwater conveyance system and can be classified into three categories:

- pervious pipe systems
- pervious catch basins
- grassed swales.

6.3.4.1 Pervious Pipe Systems

1. Purpose

Pervious pipe systems are intended to convey and infiltrate road drainage.

2. Description

Pervious pipe systems are perforated along their length, thereby promoting exfiltration of stormwater as it is conveyed downstream. The system is very similar to a conventional tile drainage system.

Pervious pipe networks are components of roadway drainage systems. Because roadway drainage usually carries a high level of suspended sediments there are associated pre-treatment components. Roadway runoff is normally directed toward grassed areas that act as sediment filters prior to flowing into the stormwater catch basin. The stormwater catch basin is raised to allow some ponding and further sediment removal. The catch basin is connected to the pervious pipe.

3. Applicability

Pervious pipe systems, although being implemented in several municipalities, are still considered experimental in nature.

4. Effectiveness

Pervious pipe systems for the exfiltration of road runoff have not proven very reliable. Pervious pipe systems experience clogging due to the high solids loads especially during construction of the pervious pipe system in new developments.

5. Water Quantity

Stormwater runoff from road surfaces contributes a substantial amount of discharge to the stormwater conveyance systems because road surfaces are normally quite impervious. Any stormwater infiltrated through the pervious pipe network reduces the total end-of-pipe discharge and therefore, any storage/treatment requirements.

6. Water Quality

Road runoff normally carries high levels of solids, oils, greases, metals, and chlorides if road salt is applied during the winter months. Removal of these contaminants prior to end-of-pipe can enhance the performance of any storage or treatment facilities. Stormwater quality can substantially improve at the end-of-pipe discharge point.

Infiltration of road runoff may, however, present a groundwater contamination problem.

7. Design Considerations

Implementation of a pervious pipe system is illustrated in Figure 6-6. Design considerations must include the pre-treatment of road runoff for solids removal. Pre-treatment can be accomplished by incorporating grassed boulevards as pre-treatment areas. To be an effective method of infiltration the surrounding soils must have a high infiltration potential. The infiltration pipe must be a sufficient height above the groundwater table to prevent groundwater from flowing into the pipe and allow for proper infiltration.

The minimum storage volume should be equal to the runoff from a 5-mm storm over the contributing drainage area. The storm volume should be accommodated in the pervious pipe bedding/storage media without overflowing. The maximum storage area should be equal to the runoff from a 25-mm storm over the contributing drainage area. The exfiltration storage bedding depth should be 75 mm to 150 mm deep above the crown of the pervious pipe and the bedding should drain 24 hours. The minimum diameter for the pervious pipe should be 200 mm and the pipe should be smooth walled to reduce the potential for clogging

6.3.4.2 Pervious Catch basins

1. Purpose

Pervious catch basins are intended to convey and infiltrate road drainage.

2. Description

Pervious catch basins are normal catch basins with larger sumps that are physically connected to an exfiltration storage media. The storage media is generally located beneath or beside the catch basin.

3. Applicability

Pervious catch basins are still considered to be experimental.

4. Effectiveness

Maintenance requirements for pervious catch basins are dependent on the clogging frequency of the infiltration media which can be high given the sediment load normally associated with road runoff. Pervious catch basins are easier to construct in new developments because they can be plugged during construction to prevent solids clogging the system.

5. Water Quantity

Stormwater runoff from road surfaces contributes a substantial amount of discharge to the stormwater conveyance systems because road surfaces are normally quite impervious. Any stormwater infiltrated through pervious catch basins reduces the total end-of-pipe discharge and therefore, any storage/treatment requirements.

6. Water Quality

Road runoff normally carries high levels of solids, oils, greases, and metals. Chlorides may also be a problem if road salt is applied during the winter months. Removal of these contaminants prior to end-of-pipe can enhance the performance of any storage or treatment facilities. Stormwater quality can substantially improve at the end-of-pipe discharge point.

7. Design Considerations

The application of a pervious catch basin for road runoff control is illustrated in Figure 6-7. The most important design consideration is the provision of adequate pre-treatment of solids to prevent frequent clogging. Design specifications recommend construction at least 1 m above the groundwater table and the use of appropriate unwoven geotextile and clear 50-mm stone to promote filtration with a low clogging frequency. To be an effective method of infiltration the surrounding soils must have a high infiltration potential. Storage volume criteria should be the same as that for pervious pipe. The depth of the exfiltration storage is dependent upon the native soil characteristics. Maximum depths can be calculated based on the native soil percolation rate. The physical dimensions of the storage will depend on the area of land available.

6.3.4.3 Grassed Swales

1. Purpose

Grassed swales store, infiltrate and convey road and on-lot stormwater runoff. Grassed swales are normally associated with more rural low-density developed drainage basins.

2. Description

Grassed swales are natural depressions or wide shallow ditches. The grass or emergent vegetation in the swale acts to reduce flow velocities, prevent erosion, and filter stormwater contaminants.

3. Applicability

Grassed swales are typically used in more rural areas with rolling or relatively flat land but can be used in place of or as an enhancement to any stormwater curb and gutter system except in strip commercial and high-density residential areas. In rural areas and in urban applications, grassed swales have been shown to effectively infiltrate runoff and remove pollutants. Grassed swales are being designed more frequently to replace curb and gutter controls and can be recommended for consideration in both rural and urban drainage basins.

4. Effectiveness

Grassed swales have been reported to provide effective quantity and quality control of urban and rural runoff. Grassed swales must be properly maintained to ensure effectiveness and prevent ponding of water. If water is allowed to pond in the swale, wetland vegetation may grow and mosquitoes may become a problem.

5. Water Quantity

Grassed swales infiltrate stormwater and reduce the end-of-pipe discharge volumes normally associated with curb and gutter controls. Significant amounts (up to 95 percent) of runoff reduction are reported in the literature pertaining to grassed swales. Grassed swales also significantly lower peak discharge rates associated with frequent storms. The changes in runoff discharge volumes and rates also reduce erosion in downstream systems.

6. Water Quality

Grassed swales can be effective in filtering and detaining stormwater runoff from a variety of catchment types. Grassed swales are effective for stormwater treatment as long as minimum channel slope is maintained and a wide bottom width is provided. Many stormwater contaminant particulates are effectively filtered by grassed swales including heavy metals, COD, nitrate nitrogen, ammonia nitrogen, and suspended solids. Other contaminant nutrients such as organic nitrogen, phosphorus, and bacteria have been reported to bypass grass swales.

7. Design Considerations

General design considerations for a grassed swale are shown in Figure 6-8. An illustration of a grassed swale with a check dam is shown in Figure 6-9.

Swales should be designed with minimum longitudinal slopes (1 to 2 percent) to promote infiltration and filtering characteristics but still maintain conveyance requirements to prevent flooding and local ponding in the swale. Check dams, as shown in Figures 6-8 and 6-9, are normally used when the longitudinal slope exceeds 2 to 4 percent. Figure 6-8 shows a perforated pipe enhancement to the swale that ensures the swale remains dry between storm events. Side slopes should be no greater than 2.5 to 1 but are optimally less than 4 to 1. A minimum bottom width of 0.75 m and minimum water depth of 0.5 m should be maintained. The maximum velocity in the swale should be 0.5 m/s. Where velocities are greater than 0.5 m/s the use of check dams (Figure 6-9) can promote infiltration and settling of pollutants. Grass should be local species or standard turf grass where a more manicured appearance is required. The grass should be allowed to grow higher than 75 mm so that suspended solids can be filtered effectively.

6.3.5 End-of-Pipe Stormwater BMPs

End-of-pipe stormwater BMPs provide water quality enhancement to stormwater prior to discharge into a receiving water body. A number of end-of-pipe alternatives are available for application depending on the characteristics of the upstream catchment and the requirements for water quality enhancement. Eight general categories of end-of-pipe BMP facilities are discussed:

- wet ponds
- dry ponds
- wetlands
- infiltration trenches
- infiltration basins
- filter strips
- sand filters
- oil/grit separators.

All references to "wet ponds", "wetlands", or "dry ponds" assume that extended detention storage is provided. Extended detention refers to the dry or active storage provided by these facilities. Extended detention ponds reduce the rate of stormwater discharge by storing the stormwater runoff temporarily and releasing it at a controlled rate. Water quality treatment is provided through enhanced settling and biological processes. As such, extended detention storage provides benefits related to water quality, erosion protection, and flooding potential.

6.3.5.1 Wet Ponds

1. Purpose

The purpose of wet ponds is to temporarily store stormwater runoff in order to promote the settlement of runoff pollutants and to restrict discharge to predetermined levels to reduce downstream flooding and erosion potentials.

2. Description

Wet ponds can be created as an impoundment by either constructing an embankment or excavating a pit. They are often designed as a two-stage (dual-purpose) facility, where the upper stage (flood fringe area) is designed to store large, infrequent storms, and the lower stage (extended detention stage) is designed to store, and promote sedimentation, of smaller, more frequent storms. The deep, permanent pond is the wet pond's primary water quality enhancement mechanism. Runoff entering the retention basin is designed to displace water already in the permanent pool and remain there until another storm event. Runoff entering the basin is slowed by the permanent pool and suspended pollutants are allowed to settle. Biologic processes, such as nutrient uptake by algae, are established in the permanent pool and help reduce concentrations of soluble contaminants. A vegetative planting strategy should provide shading, aesthetics, safety, and enhanced pollutant removal.

3. Applicability

A reliable source of runoff or groundwater discharge must be available to maintain the permanent pool of a wet pond. As such, wet ponds are generally considered for drainage areas greater than 5 ha. Because of a wet pond's ability to reduce soluble pollutants, it is generally applicable to residential, commercial, or industrial areas where nutrient loadings may be expected to be relatively high. Wet ponds may not be appropriate, or may require specialized design, where receiving water temperatures are a concern.

4. Effectiveness

Wet ponds are probably the most common end-of-pipe management facility for the control of peak runoff discharges and the enhancement of water quality. Wet ponds are very effective in controlling runoff and improving water quality when proper design considerations are made for those two objectives.

5. Water Quantity

As a detention facility, a wet pond typically flattens and spreads the inflow hygrograph, thus lowering the peak discharge. Wet ponds are effective in controlling the post-development peak discharge rate to the desired pre-development levels for design storms. Watershed/sub watershed analyses should be performed to coordinate subcatchment/pond release rates for regional flood control. Wet ponds are relatively ineffective for volume reduction, although some infiltration and/or evaporation may occur. Wet ponds are generally effective in controlling downstream erosion if designed such that the duration of post-development "critical impulses" does not exceed a pre-determined erosive threshold.

6. Water Quality

Wet ponds have been cited as providing the most reliable end-of-pipe BMP in terms of water quality treatment. This reliability is attributed to a number of factors including:

- performance does not depend on soil characteristics
- permanent pool prevents re-suspension
- permanent pool minimizes blockage of outlet
- promotes biological removal of pollutants
- permanent pool provides extended settling.

Wet ponds have a moderate to high capacity to remove most urban pollutants depending on how large the volume of the permanent pool is in relation to the runoff produced from the contributing drainage area. The establishment of vegetative zones in and around a wet pond can enhance its pollutant removal capability.

7. Design considerations

Wet ponds must be designed to meet specific water quality and/or discharge rate objectives. Wet ponds designed to control peak discharge rates do not normally provide optimum water quality enhancement. Flood control or peak flow control wet ponds are typically designed to control the large infrequent event storms. Water quality wet ponds need to be designed to capture and treat the more frequent smaller storms with which the majority of the contaminant loadings are associated. Wet ponds can be designed to meet both flood control and water quality objectives.

One of the primary criteria for the proper design of a wet pond for peak runoff control is the provision of adequate detention storage volume. The primary design consideration for a wet pond for water quality enhancement is the settling velocity of the particulates in the stormwater entering the pond. The wet pond surface area is directly related to this required settling velocity. Ponds designed only for peak flow reduction do not normally provide adequate facility for water quality enhancement.

The design of a wet pond requires careful consideration of the required design objectives for flood control and water quality enhancement. Figure 6-10 illustrates some of the basic requirements for a wet pond. Detailed designed requirements should be evaluated for each individual application based on site-specific constraints and objectives.

Some general design parameters are:

- minimum water surface area of 2 ha
- maximum side slopes above active storage zone are 4:1 to 5:1
- maximum interior side slopes in active storage zone are 5:1 to 7:1
- maximum exterior side slopes are 3:1.

Some water quality control design parameters are:

- permanent pool sized to store the volume of runoff from a 25-mm storm over the contributing area
- detention time of 24 hours
- length to width ratio shall be from 4:1 to 5:1
- minimum permanent pool depth of 2.0 m
- maximum permanent pool depth of 3.0 m. The maximum water level should be below adjacent house basement footings
- maximum active detention storage depth of 1.5 m.

Some water quantity control design parameters are:

- 1-in-100-year storm stored within 2 m above the permanent pool (Alternatively, the 2 m can be used to store the 1-in-25-year storm. In such cases an emergency overflow drainage system should be constructed with the capacity to carry storm runoff from the 1-in-100-year storm event to receiving streams or downstream stormwater management facilities.)
- Detention time of 24 hours

  Also, a wet ponds water quality control performance can be improved by providing a pre-treatment sump or forebay and a backup water supply to maintain the minimum storage volume. During the design process, other design considerations should be evaluated that relate to ease of maintenance. The forebay should be designed with the following parameters:
- Length to width ratio of 2:1 or greater

- Forebay surface area not to exceed one-third of the permanent pool surface area
- Forebay length, L_{fb} as follows:

$$L_{fb} = [rQ_p/V_s]^{0.5}$$

where:

r = Length to width ratio of forebay
Q_p = Peak flow rate from the pond during the design quality storm (m^3/s)
V_s = Settling velocity (dependent on the desired particle size to settle)

- Dispersion length, L_{dis} as follows:

$$L_{dis} = (8Q)/(dV_f)$$

where:

Q = inlet flow rate (m^3/s)
d = depth of permanent pool in the forebay (m)
V_f = desired velocity at the end of the forebay

- Forebay Bottom Width, $W = L_{dis}/8$
- Forebay berm should be 0.15 to 0.3 metres below the permanent pool elevation.

6.3.5.2 Dry Ponds

1. Purpose

The purpose of a dry pond is to temporarily store stormwater runoff in order to promote the settlement of runoff pollutants and to restrict discharge to predetermined levels to reduce downstream flooding and erosion potential.

2. Description

Dry ponds are impoundment areas constructed by an embankment or through excavating a pit. They are often designed as a two-stage (dual-purpose) facility, where the upper stage (flood fringe area) is designed to store large, infrequent storms, and the lower stage (extended detention stage) is designed to store, and promote sedimentation, of smaller, more frequent storms. Unlike a wet pond, however, the lower stage is designed to empty completely between storm events.

3. Applicability

Dry ponds may be applied where topographical or planning constraints exist that limit the land available for wet ponds. Drainage areas greater than 5 ha are generally recommended for dry ponds. The use of dry ponds for combined water quantity and quality control is discouraged without the use of sediment forebays that include a permanent pool.

A dry pond's limited effectiveness in removing soluble contaminants is an important factor in considering its application. For example, in low-density residential areas where soluble nutrients from fertilizers and pesticides are a concern, dry ponds in isolation may not be appropriate.

4. Effectiveness

Dry ponds do not provide water quality enhancement because of the bottom scour that occurs with each storm event. Dry ponds do provide effective stormwater flow attenuation.

5. Water Quantity

As a detention facility, a dry pond typically flattens and spreads the inflow hygrograph, thus lowering the peak discharge. Dry ponds are effective in controlling the post-development peak discharge rate to the desired pre-development levels for design storms. Watershed/sub watershed analyses should be performed to coordinate subcatchment/pond release rates for regional flood control. Dry ponds are relatively ineffective for volume reduction, although some evaporation may occur. Dry ponds are generally effective in controlling downstream erosion if designed such that the duration of post-development "critical impulses" does not exceed a predetermined erosive threshold.

6. Water Quality

Because dry ponds have no permanent pool of water, the removal of stormwater contaminants in dry ponds is a function of the pond's draw down time. The removal of soluble pollutants does not generally occur in a dry pond. Without a permanent pool, re-suspension of contaminants is a concern. Dry ponds operating in a continuous mode are generally less effective at pollutant removal compared to wet ponds, whereas dry ponds operating in a batch mode have been reported to be similarly effective. In general, dry ponds should only be implemented if it is determined that a wet pond cannot be implemented due to topographical or planning constraints.

7. Design Considerations

The design of a dry pond has many site-specific requirements that must be considered. These design considerations are dependent on the constraints of a particular site and the objectives for the pond.

Figure 6-11 illustrates some of the basic requirements for a dry pond.

Some general design parameters are:

- storage capacity for up to the 1-in-100-year storm
- detention time of 24 hours
- maximum active retention storage depth of 1.0 to 1.5 metres. The maximum water level should be below adjacent house basement footings.
- maximum interior side slopes of 4:1 to 5:1
- maximum exterior side slopes of 3:1
- minimum freeboard of 0.6 m

- minimum ratio of effective length to effective width of 4:1 to 5:1
- Minimum slope in the bottom of the pond of 1 percent (2 percent is preferred).

During the design process, other design considerations should be evaluated that relate to ease of maintenance and use. For example, a weeping tile system could be installed under the bottom of the pond to improve the rate at which the pond bottom dries out between storm events.

6.3.5.3 Constructed Wetlands

1. Purpose

By retaining runoff for a prolonged period of time and uptaking, altering, and storing pollutants, constructed wetlands serve to improve water quality and control peak discharge rates.

2. Description

There are five basic stormwater wetland designs: shallow marsh, pond/wetland, extended detention wetland, pocket wetland, and fringe wetland. All are essentially surface flow systems, with varying emergent marsh and deep pool habitat, hydraulic capacity, residence time, and travel routes.

Constructed wetlands can be created as an impoundment by either constructing an embankment or excavating a pit. Relatively deep permanent pools are maintained at the inlet and outlet and along low flow paths to minimize the resuspension and discharge of settled pollutants from the facility. Relatively shallow extended detention storage areas with extensive plantings (submergent and emergent) make up the majority of a constructed/artificial wetland's permanent storage. Sedimentation, filtration and biological processes account for the water quality benefits afforded by wetlands. Planting strategies are also implemented for shoreline fringe areas and/or flood fringe areas (if a combined facility) providing shading, aesthetics, safety, and enhanced pollutant removal.

3. Applicability

Generally wetlands' can be considered for drainage areas greater than 5 ha because of a wetlands ability to reduce soluble pollutants, they are generally applicable to residential, commercial, or industrial areas where nutrient loadings may be expected to be relatively high. Constructed/artificial wetlands may not be appropriate, or may require specialized design, where receiving-water temperatures are a concern. The application of constructed/artificial wetlands may be further constrained by existing planning designations or topography that limits land availability. Potential ancillary benefits provided by wetlands include aviary, terrestrial, and aquatic habitat.

Wetland water treatment systems are not recommended for all applications. Such systems are most appropriate under the following conditions:

- large tracts of suitable land are readily available
- the influent does not contain high levels of industrial toxic pollutants as defined by provincial and federal agencies
- there is a shortage of local groundwater or surface water supplies

- a water body with impaired water quality is located in the area
- the region has a history of wetland loss
- regulatory agencies are interested in the potential benefits of the technology.

4. Effectiveness

Stormwater wetland water treatment systems provide several major benefits:

- they require less maintenance and are less expensive to maintain than traditional treatment system
- with proper design, portions of the wetland treatment system may provide additional wetland wildlife habitat, as well as recreational opportunities such as bird watching, hiking, and picnicking
- wetland treatment systems are viewed as an asset by provincial and federal agencies in many regions and as a potentially effective method for replacing wetlands lost through agricultural practices, industrial and municipal development, and groundwater withdrawal.

5. Water Quantity

As a detention facility, wetlands typically flatten and spread the inflow hygrograph, thus lowering peak discharges. Wetlands are effective in controlling the post-development peak discharge rate to the desired pre-development levels for design storms. Watershed/sub watershed analyses should be performed to coordinate subcatchment/pond/wetlands release rates for regional flood control. Wetlands are relatively ineffective for volume reduction, although some infiltration and/or evaporation may occur. Wetlands are generally effective in controlling downstream erosion if designed such that the duration of post-development "critical impulses" does not exceed a predetermined erosive threshold.

6. Water Quality

In general, wetland water treatment systems have been found to lower BOD, TSS, and total nitrogen concentrations to 10 to 20 percent of the concentrations entering the systems. For total phosphorus, metals, and organic compounds, removal efficiencies vary widely, typically from 20 to 90 percent. Removal of these latter constituents appears to be limited by substrate type, the form of the constituents, the presence of oxygen, and the entire chemical makeup of the water to be treated.

7. Design Considerations

The design of a constructed wetland for dealing with urban stormwater requires a detailed study to determine from the outset what the goals of the wetland are. If the function is primarily to store water during storm events and release it later, then the size of the catchment area, permeability of the urban surfaces, and recorded flow rates will be used to determine the water volume storage capacity required. This, together with the expected frequency of large storm events, will provide an indication of the suggested draw down rates for the wetland and the diameter of outflow pipes. If, on the other hand, improving water quality is a major goal, then subsurface water flow through one or more cells may be worth incorporating into the design specifications. Should the wetland operate in the fall, winter, and early spring as well as in summer? If so, then a configuration of wetland that is deep and permits water flow during low winter temperatures may be appropriate.

Several goals may be identified for a constructed wetland, but the available site may limit the achievement of all the goals. In this case priorities must be set. The general location of a constructed wetland is an important consideration. Is it to be constructed in a residential, industrial, or rural area? Considerations such as safety, aesthetics, potential toxic spills, or wildlife mean that different design criteria must be considered. To achieve water management goals, social as well as technical issues must be addressed, for "social" problems may be more difficult to solve than physical and technical ones, and managers should involve local interest groups in the early planning stages of projects.

It is important that a pre-treatment area be provided for the collection of sediment and for the protection of the constructed wetland from accidental spills. Data is available on the construction of a pre-treatment area for oil separation and sediment removal prior to allowing water to flow into a wetland.

A constructed wetland could contain a number of cells, either of similar construction and function, or of different structure and purpose. Figure 6-12 illustrates the major components of a constructed wetland.

General design considerations are:

- wetland size should be approximately five (5) percent of the watershed area that it will be servicing
- approximately 10 percent of the wetland surface area should be a 1.5 to 2.0 m deep sediment forebay upstream of the wetland area for settleable solids removal
- average permanent water wetland depth is 0.3 m with 1 m deep zones for flow redistribution and for fish and submerged or floating aquatic vegetation habitat
- active storage is 0.3 to 0.6 m deep
- vegetation can be cost effectively transplanted from local donor sites including ditches maintained by the Province and construction sites where small pocket wetlands are to be removed
- length to width ratios can be as low as 1:1
- shape of the treatment cell(s) can vary and depends on landscaping features required for attracting wildlife and for public enjoyment, and shape of available land
- bottom slope of 0.5 to 1.0 percent is recommended and a flat bottom to promote sheet flow through the system
- gravity flow is the preferred method of movement of water into, through, and out of the treatment wetland
- incorporate a bypass that will collect first flush flows and divert high flows during extreme rainfall events around the wetland
- regulated inflow and outflow structures are required that will take into account a wide range of rainfall intensities
- landscaped features will provide an attractive park-like setting

- ancillary benefits include provision for wildlife habitat, wildlife viewing opportunities, hiking areas, educational opportunities, and restoration of lost wetland areas
- mosquito control includes introducing or making habitat available for baitfish (fathead minnows), dragonflies, purple martins, swallows, and bats
- odour control is not required since the treatment wetlands, if designed properly, do not generate odours
- nuisance wildlife including carp and muskrat will require control since they will destroy or consume the wetland vegetation and will, in the case of the carp, re-suspend settled materials
- freezing conditions during the winter months will not adversely affect the treatment wetland
- design and implement with designated objectives constantly and clearly in mind
- design more for function than for form. A number of forms can probably meet the objectives, and the form to which the system evolves may not be the planned one
- design relative to the natural reference system(s), and do not over-engineer
- design with the landscape, not against it. Take advantage of natural topography, drainage patterns, etc.
- design the wetland as an ecotone. Incorporate as much "edge" as possible, and design in conjunction with a buffer and the surrounding land and aquatic systems
- design to protect the wetland from any potential high flows and sediment loads
- design to avoid secondary environmental and community impacts
- plan on enough time for the system to develop before it must satisfy the objectives. Attempts to short-circuit ecological processes by over-management will probably fail
- design for self-sustainability and to minimize maintenance.

6.3.5.4 Infiltration Trenches

1. Purpose

The purpose of an infiltration trench is to collect and provide temporary storage of surface runoff for a specific design frequency storm and to promote subsequent infiltration. The three basic trench systems are complete exfiltration, partial exfiltration, and water quality exfiltration. Each system is defined by the volume of annual runoff diverted to the trench and the degree to which the runoff is exfiltrated into the soils. Infiltration trenches differ from on-lot infiltration systems in that they are generally constructed to manage stormwater flow from a number of lots in a developed area, not a single property.

2. Description

Infiltration trenches can be constructed at ground surface level to intercept overland flow directly, or constructed as a subsurface component of a storm sewer system. Infiltration trenches are generally composed of a clear stone storage layer and a sand or peat filter layer. There are other options for the type of filter used such as a non-woven filter fabric.

3. Application

Infiltration trenches are best utilized as recharge devices for compact residential developments (< 2 ha), rather than as a larger-scale, water quality treatment technique. Normally, infiltration trenches are not used in commercial or industrial areas because of the potential for high-contaminant loads or spills that may result in groundwater contamination.

4. Effectiveness

Infiltration trenches are effective in managing runoff from small residential areas. They are also effective when constructed under grassed swales to increase the infiltration potential of the swale. Clogging of the filter material can be a frequent problem if solids inputs are high and no pre-treatment in the form of grassed filter strip for surface trenches or a suitable oil/grit separator for subsurface trenches is employed. Groundwater mounding may also become a problem if infiltration volumes are too high.

5. Water Quantity

Infiltration trenches provide marginal water quantity control. As such, the application of infiltration trenches is likely only appropriate as a secondary facility where the maintenance of groundwater recharge is a concern.

Infiltration trenches limit the volumes of runoff normally directed toward minor drainage systems. On-lot drainage rates are also reduced. This will reduce the requirements for end-of-pipe detention storage. Effective on-lot drainage reductions on a subdivision basis will lower and flatten the receiving water inflow hydrograph.

Increased infiltration of stormwater from infiltration trenches also provides recharge to the local groundwater that may in turn discharge to local streams, thus enhancing base flows.

6. Water Quality

Pre-treatment BMPs such as filter strips or oil/water separators are often used in combination with infiltration trenches to minimize the potential for suspended sediments to clog the trench. Infiltration trenches limit the volumes of runoff from smaller storm events that are normally the major contributor of receiving water contaminants. Potential contamination of groundwater should be considered when examining runoff quality directed to the infiltration trench.

7. Design Considerations

A surface infiltration trench and a subsurface infiltration trench are shown in Figures 6-13 and 6-14, respectively. Infiltration trenches require groundwater levels and bedrock layers to be at least 1 m below the bottom of the infiltration trench. Soils must have a percolation rate of more that 15 mm/hr. A suitable filter fabric should be used to protect the stone storage media from clogging.

Careful consideration should be given to the volume of stormwater directed to the infiltration trench. Only sufficient volumes should be directed to the trench to allow, at a maximum, a 48-hour draw down period.

In a subsurface trench, a series of perforated pipes carries stormwater to the trench. A bypass pipe or flow path should be provided for flows in excess of the design capacity of the trench.

6.3.5.5 Infiltration Basins

1. Purpose

The purpose of an infiltration basin is to collect and provide temporary storage of surface runoff for a specific design frequency storm and to promote subsequent infiltration.

2. Description

Infiltration basins are aboveground pond impoundment systems that promote recharge. Water percolating through an infiltration basin either recharges the groundwater system or is collected by an underground-perforated pipe system and discharged at a downstream outlet. The appearance of an infiltration basin is similar to that of a wet or dry pond.

3. Applicability

Infiltration basins are generally considered for drainage areas less than 5 ha that have permeable soils. As with wet or dry ponds, an infiltration basin can be designed as a multi-stage facility to achieve various stormwater management objectives. Infiltration basins should be used in residential areas only. Runoff from industrial or commercial land areas is generally of poor quality and could contaminate groundwater.

4. Effectiveness

Infiltration basins have a very high rate of failure. Most failures can be attributed to poor site selection, poor design, poor construction techniques, large drainage area, and lack of maintenance. One of the main problems inherent in infiltration basins is that large volumes of water from a large catchment area are expected to infiltrate over a very small surface area. This leads to numerous problems and general failure of these basins.

5. Water Quantity

Infiltration basins are generally ineffective for water quantity control. They only infiltrate limited volumes of water from generally large catchment areas and must be provided with an overflow structure to discharge excess flow. As such, the application of infiltration basins is likely only appropriate as a secondary facility where the maintenance of groundwater recharge is a concern.

6. Water Quality

The application of pre-treatment to reduce sediment loadings and a bypass to restrict flows during certain periods (road sanding/salting, local excavation works, facility maintenance) is recommended to improve long-term infiltration basin performance.

7. Design Considerations

A typical infiltration basin is illustrated in Figure 6-15. Infiltration basin design considerations must include provision for construction at the end of the development construction. Also, compaction of the basin and smearing of the basin native material must be avoided. The basin must be constructed with a maximum water storage depth of 0.6 m to avoid compaction, and the groundwater table should be a minimum of 1.0 m below the infiltration layer. Any area bedrock should also be a minimum of 1.0 m below the infiltration layer. Planting in the basin should include grasses and legumes to maintain or enhance the pore spaces in the soil.

6.3.5.6 Filter Strips

1. Purpose

Filter strips are engineered conveyance systems that are designed to remove pollutants from overland runoff. By reducing overland flow velocities, the time of concentration and infiltration are increased, thereby slightly reducing the volume of runoff and minimally controlling discharge rates.

2. Description

There are two general types of filter strips: grass and forested. Both consist of a level spreader, which ensures level flows, and abundant vegetative plantings. The vegetative plantings promote pollutant filtration and infiltration of stormwater. Filter strips are generally best implemented adjacent to a buffer strip, watercourse, or drainage swale, as discharge from a filter strips will be a sheet flow and thus difficult to convey in a traditional stormwater conveyance system.

3. Applicability

Filter strips are best applied as one of a combination of BMPs as the maintenance of sheet flow through the vegetation, and thus consistent water quality benefits, has been difficult to maintain in practice.

4. Effectiveness

Limited filter strip performance data are available in the literature although it is generally thought that properly designed filter strips are capable of removing a high percentage of stormwater particulates.

5. Water Quantity

Filter strips may slightly reduce the volume of runoff by inducing infiltration.

6. Water Quality

Although filter strips have been shown to be somewhat effective in removing sediment and pollutant loads in urban stormwater runoff, the ability to maintain sheet flow through the vegetation over the long term has been questioned.

7. Design Considerations

A schematic of a grassed and wooded filter strip is shown in Figure 6-16. The filter strip requires a level spreader with available upstream storage to regulate the discharge rate and depth of flow through the filter strip. The ideal slope for a filter strip is less than 5.0 percent over a distance of 10 to 20 m in the direction of flow.

6.3.5.7 Sand Filters

1. Purpose

Sand filters are above or below ground end-of-pipe treatment devices that promote pollutant removal from overland runoff or storm sewer systems. Sand filters do not provide a recharge benefit as filtered stormwater is discharged to the storm sewer or receiving water.

2. Description

Sand filters can be constructed either above or below ground as an end-of-pipe BMP. They are most commonly constructed with impermeable liners to guard against native material clogging pore spaces and to prevent filtered water from entering the groundwater system. Water that infiltrates through the filer is collected by a previous pipe system and conveyed to a downstream outlet. Some designs incorporate a layer of peat to enhance pollutant removal capabilities of the sand filter, thus making discharge to an infiltration trench a possibility.

3. Applicability

Sand filters can be constructed either above or below ground as an end-of-pipe BMP and are generally only appropriate for relatively small drainage areas (< 5 ha). Also, very little is known in regard to sand filter performance and cold-climate operation and maintenance.

4. Effectiveness

This method of water quality enhancement should not be generally applied without a detailed feasibility assessment.

5. Water Quantity

Sand filters are not suitable for water quantity control as they should not be designed to handle large influent flows.

6. Water Quality

Sand filters have been found to be effective in removing pollutants, however, little is know about their performance in winter or freshet conditions.

7. Design Considerations

A sand filter application is illustrated in Figure 6-17. Sand filters can be constructed as surface filters or subsurface filters as part of the stormwater conveyance system. Surface filters are normally covered by a grass layer. Filters are lined with impermeable membranes to restrict clogging of the filter material by native material.

6.3.5.8 Oil/Grit Separators

1. **Purpose**

Oil/grit separators are a variation of traditional settling tanks. They are designed to capture sediment and trap hydrocarbons suspended in runoff from impervious surfaces as the runoff is conveyed through a storm sewer network.

2. **Description**

An Oil/grit separator is a belowground, pre-cast concrete structure that takes the place of a conventional manhole in a storm drain system. The separator implements the use of permanent pool storage in the removal of hydrocarbons and sediment from stormwater runoff before discharging into receiving waters or storm sewer systems.

3. **Applicability**

Oil/grit separators are typically applied to urban-based drainage areas (<5ha) where ponds or wetlands are not feasible or cost effective. Separators are best applied in areas of high impervious cover where there is a potential for hydrocarbon spills and polluted sediment discharges. Typical applications include parking lots, commercial & industrial sites, petroleum service stations, airports, and residential developments (pre-treatment of ponds/wetlands or as part of a treatment train).

4. **Effectiveness**

Oil/grit separators can be effective for treatment of stormwater pollution at its source. Source control is favorable for water quality control since the dilution of pollutants in stormwater becomes problematic in terms of effective treatment as the drainage area increases. Depending on land use, drainage area, site conditions, and hydrology, some oil/grit separators may be effective in reducing TSS. See Table 6-2 for oil/grit separator design types and characteristics.

5. **Water Quantity**

Oil/grit separators implement the use of permanent pool storage for removal of stormwater pollution. However, they are not designed to provide extended detention storage, and thus provide little flow attenuation.

6. **Water Quality**

Oil/grit separators vary in design and performance. Separators that do not incorporate a high flow bypass have been found to be generally ineffective in removing/containing hydrocarbon and sediment pollutants, because of a continuous process of re-suspension and settling of solids.

7. **Design Considerations**

Three chambered oil/grit separators operate most effectively when constructed offline. A flow splitter should be used to direct excess flow back to the conveyance system or to some other control practice. Only low flows should be directed to the separator.

Bypass separators are installed online, and high flows do not affect the performance of the unit.

See Figures 6-18 and 6-19 for illustrations of the two types of oil/grit separators.

6.3.6 BMP Screening and Selection

6.3.6.1 Initial Screening

There are a range of stormwater BMP options available for most applications. The selection of an appropriate BMP or group of BMPs depends first on the objectives for stormwater management defined for a particular catchment area, as well as the constraints placed on the feasibility of particular BMPs by physical site factors.

Once the objectives for stormwater management are well defined and the site constraints are understood individual BMPs can be evaluated in terms of their overall effectiveness as a stormwater control facilities. The evaluation of overall effectiveness must include both water quantity and water quality objectives.

Also, each stormwater management BMP has associated with it certain advantages and disadvantages that may allow the viable options for stormwater management to be reduced for a particular development area.

Table 6-2 summarizes the advantages and disadvantages of a number of BMPs.

6.3.6.2 Physical Constraints

Site characteristics may be the factor that will ultimately determine the applicability of individual or combinations of BMPs. Physical factors that need to be assessed in evaluating the suitability of BMPs include:

- topography
- soils stratification
- depth to bedrock
- depth to seasonably high water table
- drainage area.

Table 6-3 summarizes physical constraints associated with various BMP types.

6.3.6.3 Final Screening

In the initial screening phase the options for BMPs were limited by particular disadvantages and site constraints. The list of BMP options that are still considered feasible are further screened by the application of specific objectives that must be met as part of the development including:

- water quality
- flooding
- erosion
- recharge.

The performance of the BMPs in regard to the objectives for stormwater management are shown in Table 6-4.

6.3.6.4 Water Quality Control and Enhancement Opportunities

In many areas of development, stormwater management practices must meet stringent water quality objectives to protect sensitive receiving waters. Water quality objectives can be defined for a stormwater management system and then appropriate BMPs can be selected from the pre-screened list that will meet the water quality objectives.

The reported effectiveness, to remove pollutants, of a number of BMPs are shown in Table 6-5.

TABLE 6-2
BMP ADVANTAGES AND DISADVANTAGES

BMP	Advantages	Disadvantages
Wet pond	· Capable of removing soluble as well as solid pollutants · Provides erosion control · Habitat, aesthetic, and recreation opportunities provided · Relatively less frequent maintenance schedule	· More costly than dry ponds · Permanent pool storage requires larger land area · Could have negative downstream temperature impacts · Could be constrained by topography or land designations · Sediment removal relatively costly when required
Dry pond	· Batch mode has comparable effectiveness to wet ponds · Not constrained by land area required by wet ponds · Can provide recreational benefits	· Potential re-suspension of contaminants · More expensive O&M costs than wet ponds (batch mode)
Wetlands	· Pollutant-removal capability similar to wet ponds · Offers enhanced nutrient-removal capability · Potential ancillary benefits, including aviary, terrestrial, and aquatic habitat	· Requires more land area than wet ponds · Could have negative downstream temperature impacts · Could be constrained by topography or land designations · Potential for some nuisance problems
Infiltration trenches	· Potentially effective in promoting recharge and maintaining low flows in small areas · May be appropriate as secondary facility where maintenance of groundwater recharge is a concern · No thermal impact · No public safety concern	· Appropriate only to small drainage areas (<2 ha) and residential land uses · Constrained by native soil permeability's · Usually requires pre-treatment device · Potential contamination of groundwater must be investigated · Generally ineffective for water quantity control · High rate of failure due to improper siting and design, pollutant loading, and lack of maintenance
Infiltration basins	· Potentially effective in promoting recharge and maintaining low flows in small areas · May be appropriate as secondary facility where maintenance of groundwater recharge is a concern · No thermal impact · No public safety concern	· Appropriate only to relatively small drainage areas (<5 ha) and residential land uses · Constrained by native soil permeability's · Pre-treatment is recommended · Potential contamination of groundwater must be investigated · Generally ineffective for water quantity control · High rate of failure due to improper siting and design, pollutant loading, and lack of maintenance

TABLE 6-2
BMP ADVANTAGES AND DISADVANTAGES

BMP	Advantages	Disadvantages
Filter strips	· Water quality benefits may be realized if part of overall SUM plan (i.e., as secondary facility) · Effective in filtering out suspended solids and intercepting precipitation · May reduce runoff by reducing overland flow velocities, increasing time of concentration, and increasing infiltration · Can create wildlife habitat · No thermal impact	· Limited to small drainage areas (<2 ha) with little topographic relief · Uniform sheet flow through vegetation difficult to maintain · Effectiveness in freeze/thaw conditions questionable
Sand filters	· Generally effective in removing pollutants, are resistant to clogging and are easier/less expensive to retrofit compared to infiltration trenches	· Not suitable for water quantity control · Generally applicable to only small drainage areas (<5 ha) · Do not generally recharge groundwater system · May cause aesthetic/odour problems · O&M costs generally higher than other end-of-pipe facilities
Oil/Grit Separators (3-Chamber Separator)	· Offline, 3-chamber (oil, grit, discharge) separators may be appropriate for commercial, industrial, large parking, or transportation-related areas less than 2 ha	· Scour and resuspension of trapped pollutants in heavy rainfall events · Difficult to maintain · Relatively high O&M costs · Online design of 3-chamber separators has resulted in poor pollutant removal performance
Oil/Grit Separators (Bypass Separator)	· Bypass prevents the scouring and resuspension of trapped pollutants in heavy rainfall events · Effective in removing sediment load when properly applied as a source control for small areas · Effective in trapping oil/grease from run off.	· Relatively high capital costs compared to manholes · Applicable for drainage areas less than 5 ha.

TABLE 6-3
PHYSICAL BMP CONSTRAINTS

BMP	Criteria				
	Topography	Soils	Bedrock	Groundwater	Area
On-Lot BMP					
Flat lot grading	<5%	none	none	none	none
Soak-away pit	none	loam (min. infiltration rate ≥15 mm/h)	>1 m below bottom	>1 m below bottom	<0.5 ha
Rear yard infiltration	<2%	loam (min. infiltration rate ≥15 mm/h)	>1 m below bottom	>1 m below bottom	<0.5 ha
Conveyance BMP					
Grassed swales	<5%	none	none	none	none
Perforated pipes	none	loam (min. infiltration rate ≥15 mm/h)	>1 m below bottom	>1 m below bottom	none
Pervious catch basins	none	loam (min. infiltration rate ≥15 mm/h)	>1 m below bottom	>1 m below bottom	none
End-of-Pipe BMP					
Wet pond	none	none	none	none	>5 ha
Dry pond	none	none	none	none	>5 ha
Wetland	none	none	none	none	>5 ha
Infiltration basin	none	loam (min. infiltration rate ≥15 mm/h)	>1 m below bottom	>1 m below bottom	<5 ha
Infiltration trench	none	loam (min. infiltration rate ≥15 mm/h)	>1 m below bottom	>1 m below bottom	<2 ha
Filter strips	<10%	none	none	>0.5 m below bottom	<2 ha
Sand filters	none	none	none	>0.5 m below bottom	<5 ha
Oil/grit separators	none	none	none	none	<1 ha

TABLE 6-4
POTENTIAL BMP OPPORTUNITIES

Stormwater BMP	Water Quality	Flooding	Erosion	Recharge
Lot Level BMPs				
Lot grading	♦	♦	♦	•
Roof leader ponding	♦	♦	♦	•
Roof leader soak-away pits	♦	♦	♦	•
Conveyance BMPs				
Pervious pipes	•*	♦	♦	•
Pervious catch basins	•*	♦	♦	•
Grassed swales	•	♦	•	♦
End-of-Pipe BMPs				
Wet pond	•	•	•	❍
Dry pond	♦	❍	•	❍
Dry pond with forebay	•	•	•	❍
Wetland	•	•	•	❍
Sand filter	•	♦	♦	❍
Infiltration trench	♦*	♦	♦	•
Infiltration basin	♦*	♦	♦	•
Vegetated filter strip	•	❍	♦	♦
Buffer strip	♦	❍	♦	♦
Special purpose BMP				
Oil/grit separator	♦	❍	❍	❍

• Highly effective (primary control)
♦ Limited effectiveness (secondary control)
❍ Not effective
* May have adverse effects
From MOEE, 1994

TABLE 6-5
EFFECTIVENESS OF BEST MANAGEMENT PRACTICES FOR CONTROL OF RUNOFF FROM NEWLY DEVELOPED AREAS

Management Practice		Removal Efficiency (%)						Factors	References
		TSS	TP	TN	COD	Pb	Zn		
Infiltration Basin	Average:	75	65	80	65	65	65	Soil percolation rates Basin surface area Storage volume	NVPDC, 1979; EPA, 1977; Schueler, 1967; Griffin et al, 1980; EPA, 1963; Woodword-Clyde, 1966
	Reported Range:								
	SCS Soil Group A	60-100	60-100	60-100	60-100	60-100	60-100		
	SCS Soil Group B	50-80	50-80	50-80	50-80	50-80	50-80		
	No. of Values Considered:	7	7	7	4	4	4		
Infiltration Trench	Average:	75	60	55	65	65	65	Soil Percolation rates Trench surface area Storage volume	NVPDC, 1979; EPA, 1977; Schueler, 1967; Griffin et al, 1980; EPA, 1963; Woodword-Clyde, 1966; Kuo et al 1968; Lugbill, 1990
	Reported Range:	45-100	40-100	(110)-100	45-100	45-100	45-100		
	Probable Range:								
	SCS Soil Group A	60-100	60-100	60-100	60-100	60-100	60-100		
	SCS Soil Group B	50-90	50-90	50-90	50-90	50-90	50-90		
	No. of Values Considered:	9	9	9	4	4	4		

TABLE 6-5
EFFECTIVENESS OF BEST MANAGEMENT PRACTICES FOR CONTROL OF RUNOFF FROM NEWLY DEVELOPED AREAS

Management Practice		Removal Efficiency (%)						Factors	References
		TSS	TP	TN	COD	Pb	Zn		
Vegetated Filter Strip	Average:	65	40	40	40	45	60	Runoff volume	IEP, 1991 Casman, 1990 Glick et al, 1991 VADC, 1987 Minnesota PCA, 1989 Scheuler, 1967 Hartigan et al 1969
	Reported Range:	20-80	0-95	0-70	0-60	20-90	30-90	Slope	
	Probable Range:	40-90	30-80	20-60	-	30-80	20-50	Soil infiltration rates	
	No. of Values Considered:	7	4	3	2	3	3	Vegetative cover Buffer length	
Grass Swale	Average:	60	20	10	25	70	60	Runoff volume	Yousel et al, 1965 Dupuls, 1985 Washington State, 1968 Schuerer, 1967 British Columbia Res. Corp, 1991 EPA, 1983 Whelen et al, 1988 PIN, 1966 Caeman, 1990
	Reported Range:	0-100	0-100	0-40	25	3-100	50-80	Slope	
	Probable Range:	20-40	20-40	10-30	-	10-20	10-20	Soil infiltration rates	
	No. of Values Considered	10	8	4	1	10	7	Vegetative cover Swale length Swale geometry	

TABLE 6-5
EFFECTIVENESS OF BEST MANAGEMENT PRACTICES FOR CONTROL OF RUNOFF FROM NEWLY DEVELOPED AREAS

Management Practice		Removal Efficiency (%)						Factors	References
		TSS	TP	TN	COD	Pb	Zn		
Porous Pavement	Average:	35	5	20	5	15	5	Maintenance	Pitt, 1965 Field, 1985 Schueler, 1967
	Reported Range:	0-95	5-10	5-55	5-10	10-25	5-10	Sedimentation storage volume	
	Probable Range:	10-25	5-10	5-10	5-10	10-25	5-10		
	No. of Values Considered:	3	1	2	1	2	1		
Concrete Grid Pavement	Average:	90	90	90	90	90	90	Percolation rates	Day, 1961 Smith et al, 1961 Schueler, 1967
	Reported Range:	65-100	65-100	65-100	65-100	65-100	65-100		
	Probable Range:	60-90	60-90	60-90	60-90	60-90	60-90		
	No. of Values Considered:	2	2	2	2	2	2		
Sand Filter/ Filtration Basin	Average:	80	50	35	55	60	65	Treatment volume	City of Austin, 1986 Environmental and Conservation Service Department, 1990
	Reported Range:	60-95	0-90	20-40	45-70	30-90	50-80	Filtration media	
	Probable Range:	60-90	0-80	20-40	40-70	40-80	40-80		
	No. of Values Considered:	10	6	7	3	5	5		

TABLE 6-6
EFFECTIVENESS OF BEST MANAGEMENT PRACTICES FOR CONTROL OF RUNOFF FROM NEWLY DEVELOPED AREAS

Management Practice		Removal Efficiency (%)						Factors	References
		TSS	TP	TN	COD	Pb	Zn		
Water Quality Inlet	Average:	35	5	20	5	15	5	Maintenance Sedimentation storage volume	Pitt, 1965 Field, 1965 Schueler, 1967
	Reported Values:	0-95	5-10	5-55	5-10	10-25	5-10		
	Probable Values:	10-25	5-10	5-10	5-10	10-25	5-10		
	No. of Values Considered:	3	1	2	1	2	1		
Water Quality Inlet with Sand Filter	Average:	80	NA	35	55	80	65	Sedimentation storage volume Depth of media	Shaver, 1991
	Reported Range:	75-85	NA	30-45	45-70	70-90	50-80		
	Probable Range:	70-90	_	30-40	40-70	70-90	50-80		
	No. of Values Considered:	1	0	1	1	1	1		
Oil/Grit Separator	Average:	15	5	5	5	15	5	Sedimentation storage volume Outlet configurations	Pitt, 1965 Schueler, 1967
	Reported Range:	0-25	5-10	5-10	5-10	10-25	5-10		
	Probable Range:	10-25	5-10	5-10	5-10	10-25	5-10		
	No. of Values Considered:	2	1	1	1	1	1		

TABLE 6-5
EFFECTIVENESS OF BEST MANAGEMENT PRACTICES FOR CONTROL OF RUNOFF FROM NEWLY DEVELOPED AREAS

Management Practice		Removal Efficiency (%)						Factors	References
		TSS	TP	TN	COD	Pb	Zn		
Extended-Detention Dry Pond	Average:	45	25	30	20	50	20	Storage volume Detention time Pond shape	MWCOG, 1983 City of Austin, 1990 Schueler and Heinrich, 1965 Pope and Hess, 1989 OWML, 1967 Wollnold and Stack, 1990
	Reported Range:	5-90	10-55	20-60	0-40	25-65	(-40)-65		
	Probable Range:	70-90	10-60	20-60	30-40	20-60	40-60		
	No. of Values Considered:	6	6	4	5	4	5		
Wet Pond	Average:	60	45	35	40	75	80	Pond volume Pond shape	Wotzka and Oberta, 1966 Yousel et al, 1968 Cullum, 1985 Driscoll, 1983 Driscoll, 1986 MWCOG, 1963 OWML, 1963 Yu and Benemouflok, 1986 Hother, 1989 Martin, 1966 Dowman et al, 1969 OWML, 1962 City of Austin, 1990
	Reported Range:	(-30)-91	10-85	5-85	5-90	10-85	10-95		
	Probable Range:	50-90	20-90	10-90	10-90	10-95	20-95		
	No. of Values Considered:	18	18	9	7	13	13		

TABLE 6-5
EFFECTIVENESS OF BEST MANAGEMENT PRACTICES FOR CONTROL OF RUNOFF FROM NEWLY DEVELOPED AREAS

Management Practice		Removal Efficiency (%)						Factors	References
		TSS	TP	TN	COD	Pb	Zn		
Extended-Detention Wet Pond	Average:	80	65	55	NA	40	20	Pond volume Pond shape Detention time	Ontario Ministry of the Environment, 1991 cited in Schueler et al 1992
	Reported Range:	50-100	50-60	55	NA	40	20		
	Probable Range:	50-95	50-90	10-90	10-90	10-95	20-95		
	No. of Values Considered:	3	3	1	0	1	1		
Constructed Stormwater Wetlands	Average:	65	25	20	50	65	35	Storage volume Detention time Pool shape Wetlands biota Seasonal variation	Harper et al, 1966 Brown, 1985 Wotzka and Oberta, 1966 Hickock et al, 1977 Burten, 1967 Martin, 1966 Morris et al, 1961 Sherberger and Davis, 1962 ABAG, 1979 Oberts et al, 1969 Rushton and Dye, 1990 Hay and Barrett, 1991 Martin and Smool, 1986 Ralnelt et al, 1990 cited in Woodward and Clyde, 1991
	Reported Range:	(-20)-100	(-120)-100	(-15)-40	20-80	30-95	(-30)-60		
	Probable Range:	50-90	(-5)-80	0-40	-	30-95	-		
	No. of Values Considered:	23	24	6	2	10	8		

NA - Not available

[a] Design criteria: storage volume equals 80% avg. runoff volume, which completely drains in 72 hours; maximum depth = 6 ft.; minimum depth = 2 ft.

[b] Design criteria: storage volume equals 90% avg. runoff volume, which completely drains in 72 hours; maximum depth = 5 ft.; minimum depth = 3 ft.; storage volume = 40% excavated trench volume

[c] Design criteria: flow depth < 0.3 ft.; travel time > 5 min.

[d] Design criteria: Low slope and adequate length

[e] Design criteria: minimum extended detention time 12 hours

[f] Design criteria: minimum area of wetland equal 1% of drainage area

[g] No information was available on the effectiveness of removing oil and grease

[h] Also reported as 90% TSS removed

[i] Also reported as 50% TSS removed

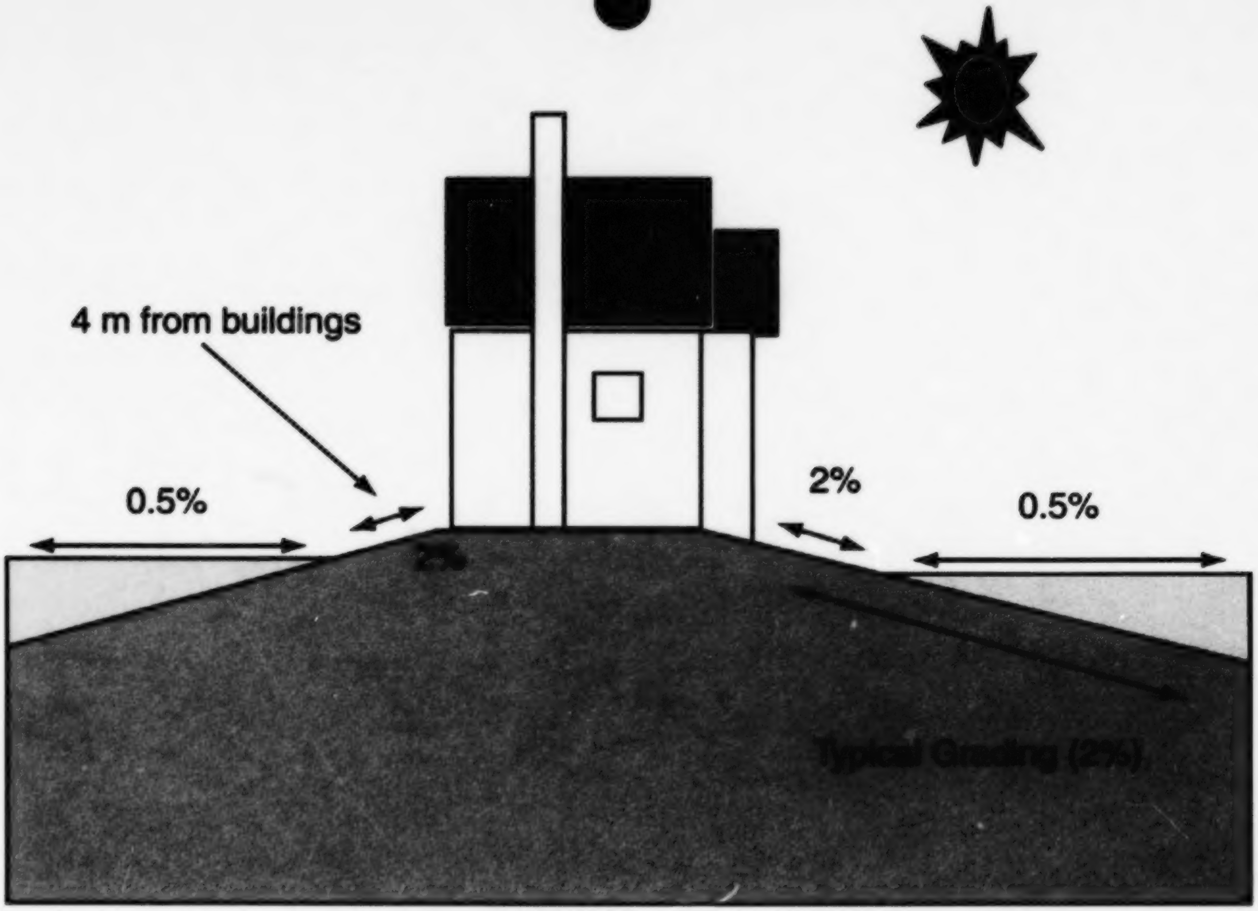

4 m from buildings
0.5%
2%
0.5%
Typical Grading (2%)

Figure 6.1
Lot Grading Guidelines

Rear Yard Ponding

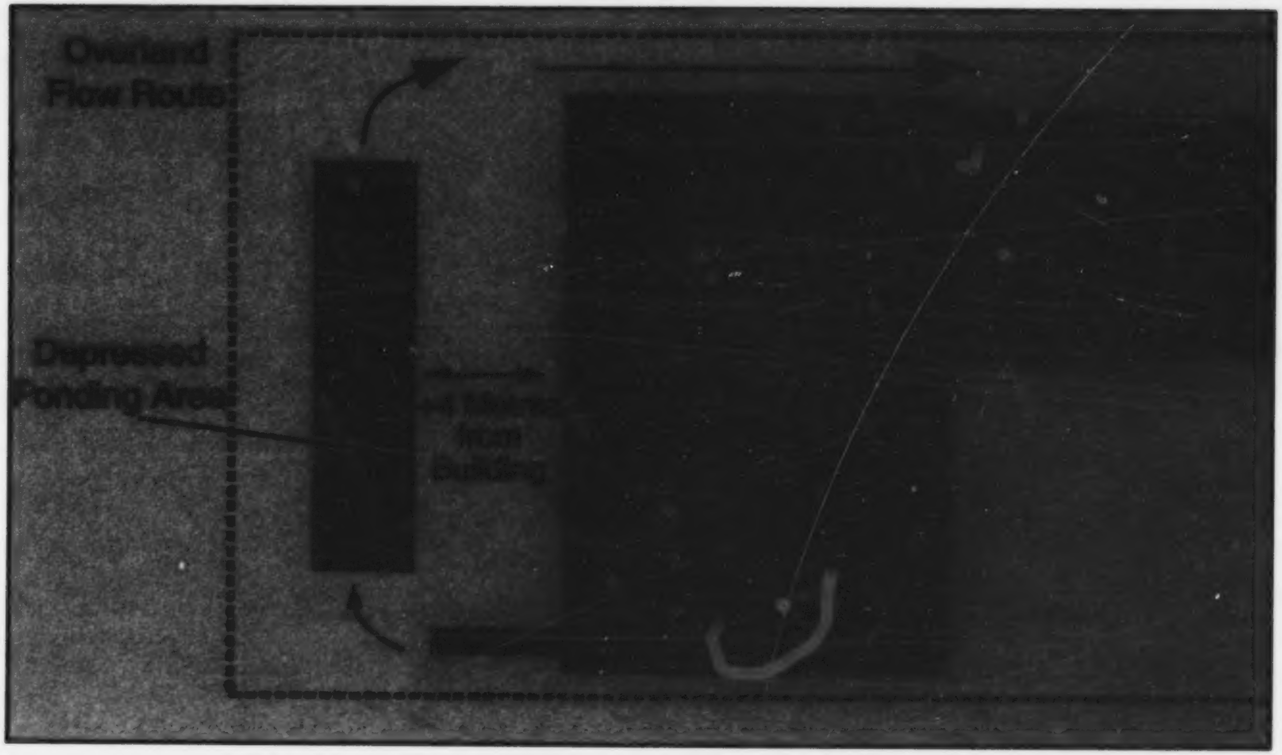

Overland
Flow Route
Depressed
Ponding Area
4 Metres
from
Building

Figure 6.2

Rear Lot Ponding Guidelines

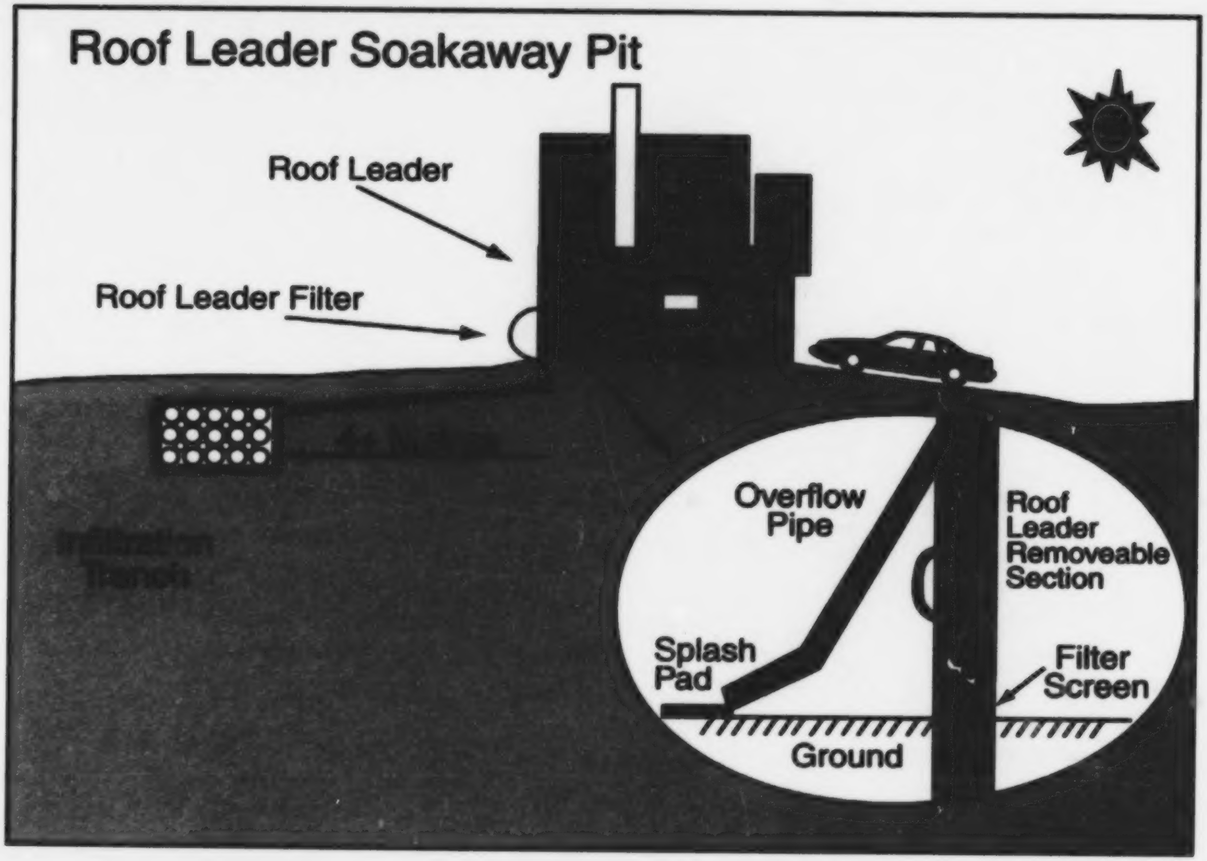
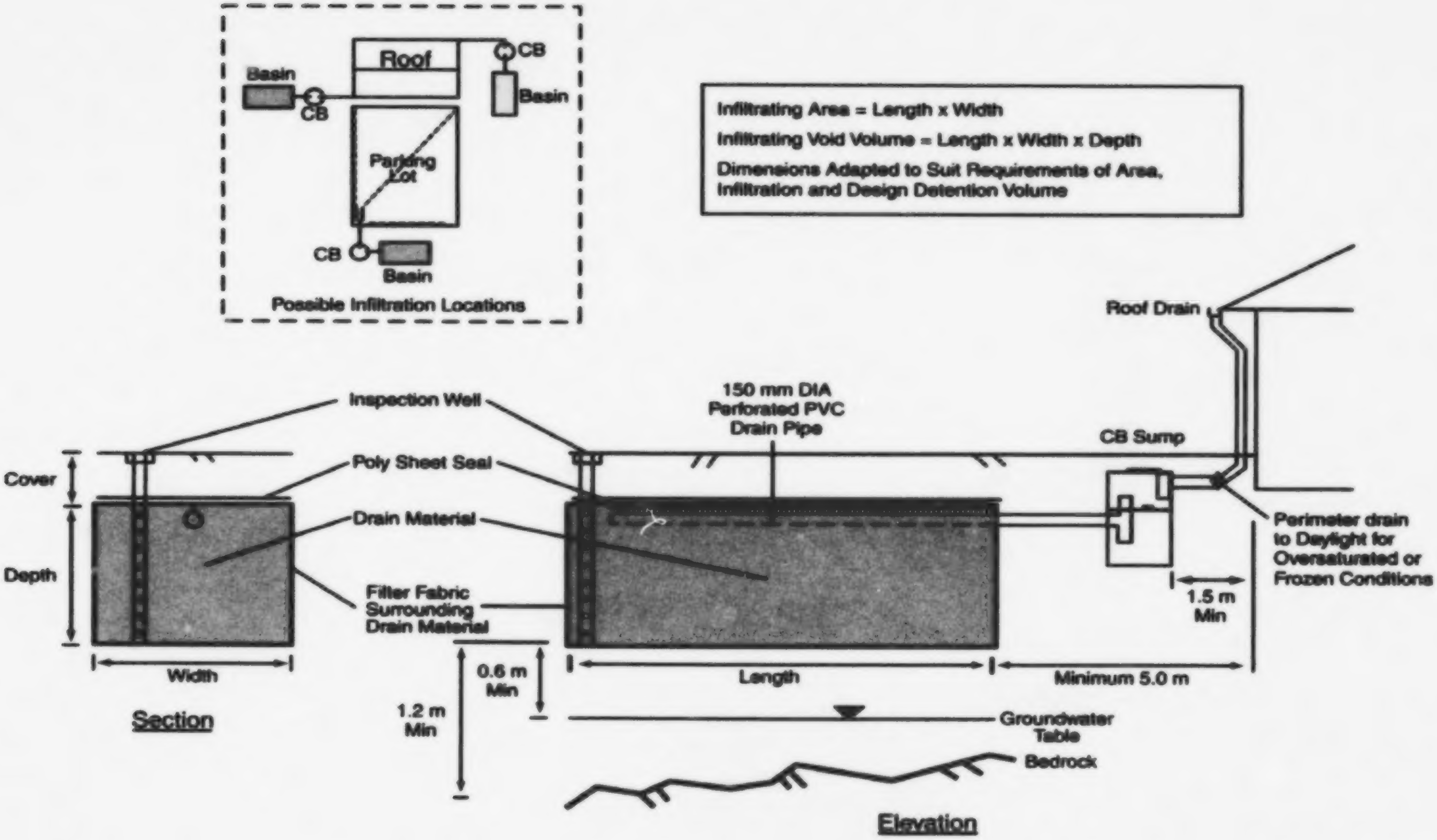

Roof Leader Soakaway Pit
Roof Leader
Roof Leader Filter
Infiltration Trench
Overflow Pipe
Roof Leader Removeable Section
Splash Pad
Filter Screen
Ground

Figure 6.3

Infiltration System with Roof Leader Filter


Roof
CB
Basin
Basin
CB
Parking Lot
CB
Basin
Possible Infiltration Locations
Roof Drain
Inspection Well
150 mm DIA Perforated PVC Drain Pipe
CB Sump
Cover
Poly Sheet Seal
Drain Material
Depth
Filter Fabric Surrounding Drain Material
Perimeter drain to Daylight for Oversaturated or Frozen Conditions
1.5 m Min
Width
0.6 m Min
Length
Minimum 5.0 m
Section
1.2 m Min
Groundwater Table
Bedrock
Elevation

Infiltrating Area = Length x Width

Infiltrating Void Volume = Length x Width x Depth

Dimensions Adapted to Suit Requirements of Area, Infiltration and Design Detention Volume

Figure 6.4
Infiltration System with Pretreatment Sump

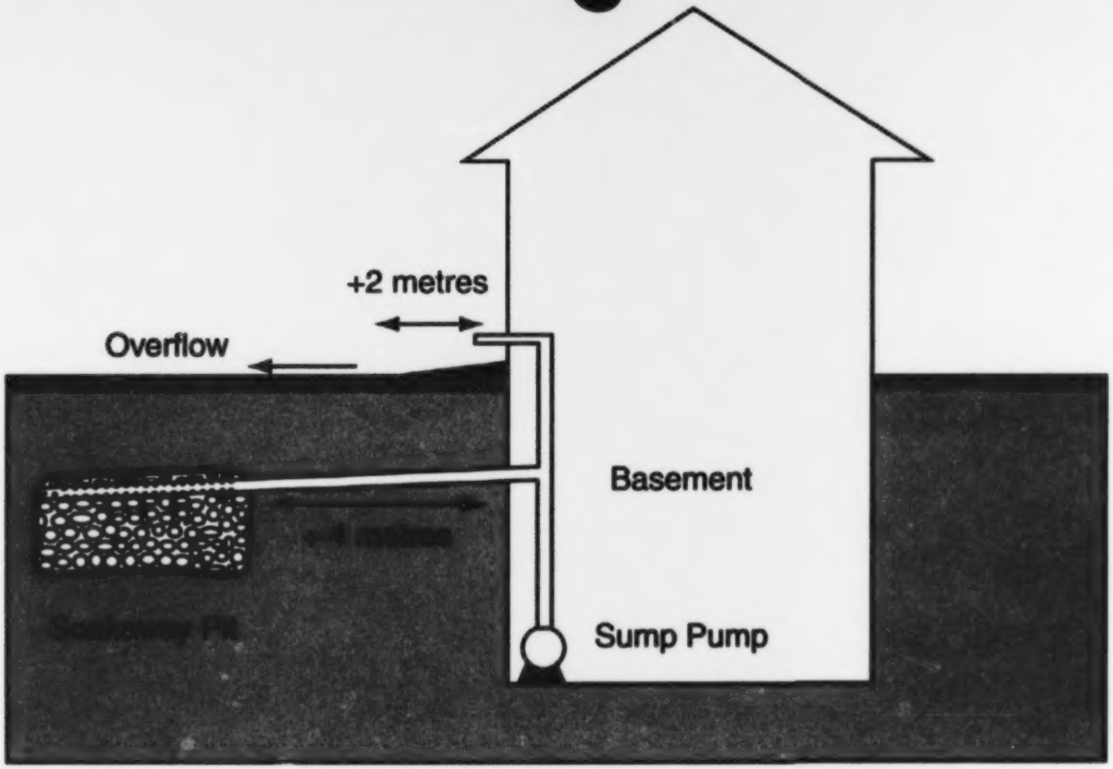

+2 metres
Overflow
Basement
+4 metres
Soakaway Pit
Sump Pump

Figure 6.5

Sump Pump Foundation Drainage

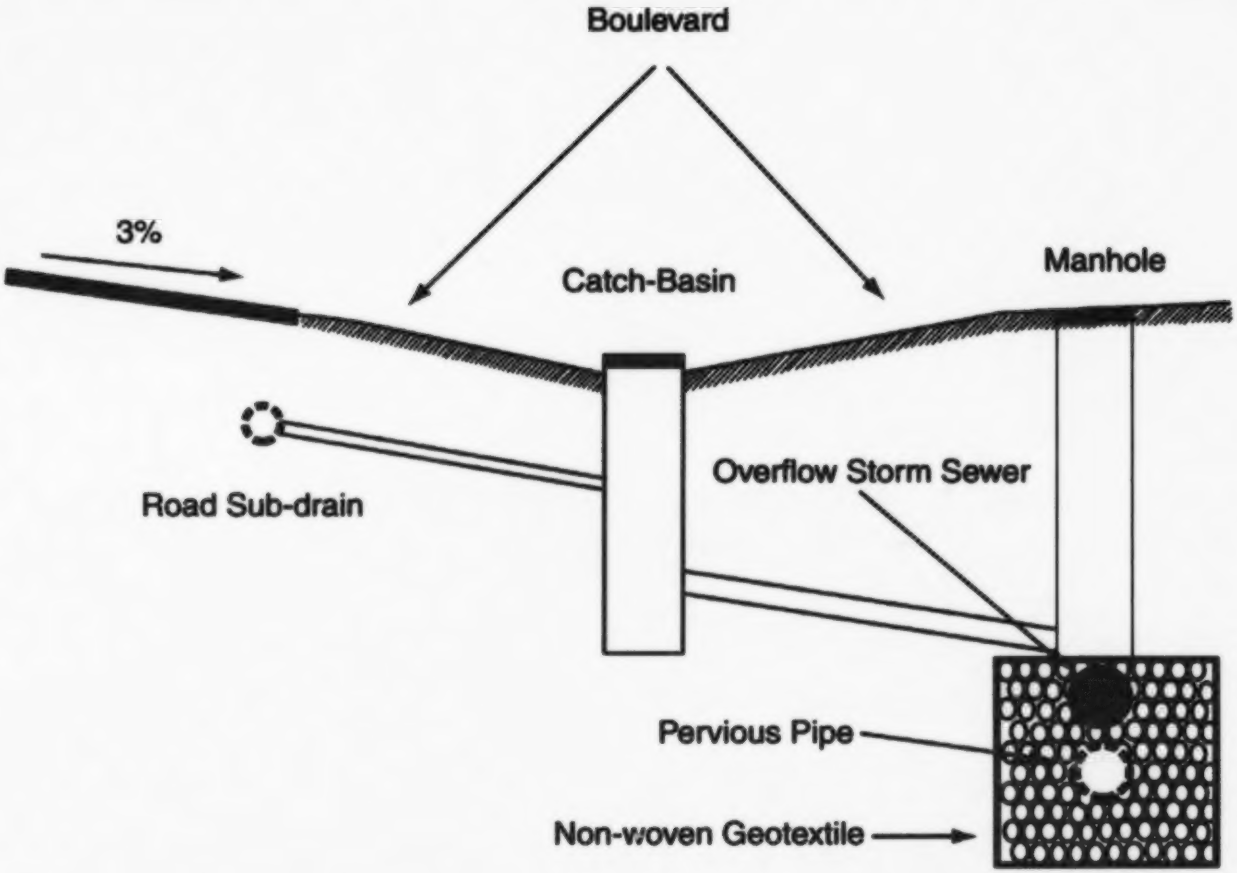

Boulevard
3%
Catch-Basin
Manhole
Road Sub-drain
Overflow Storm Sewer
Pervious Pipe
Non-woven Geotextile

Figure 6.6

Pervious Pipe System

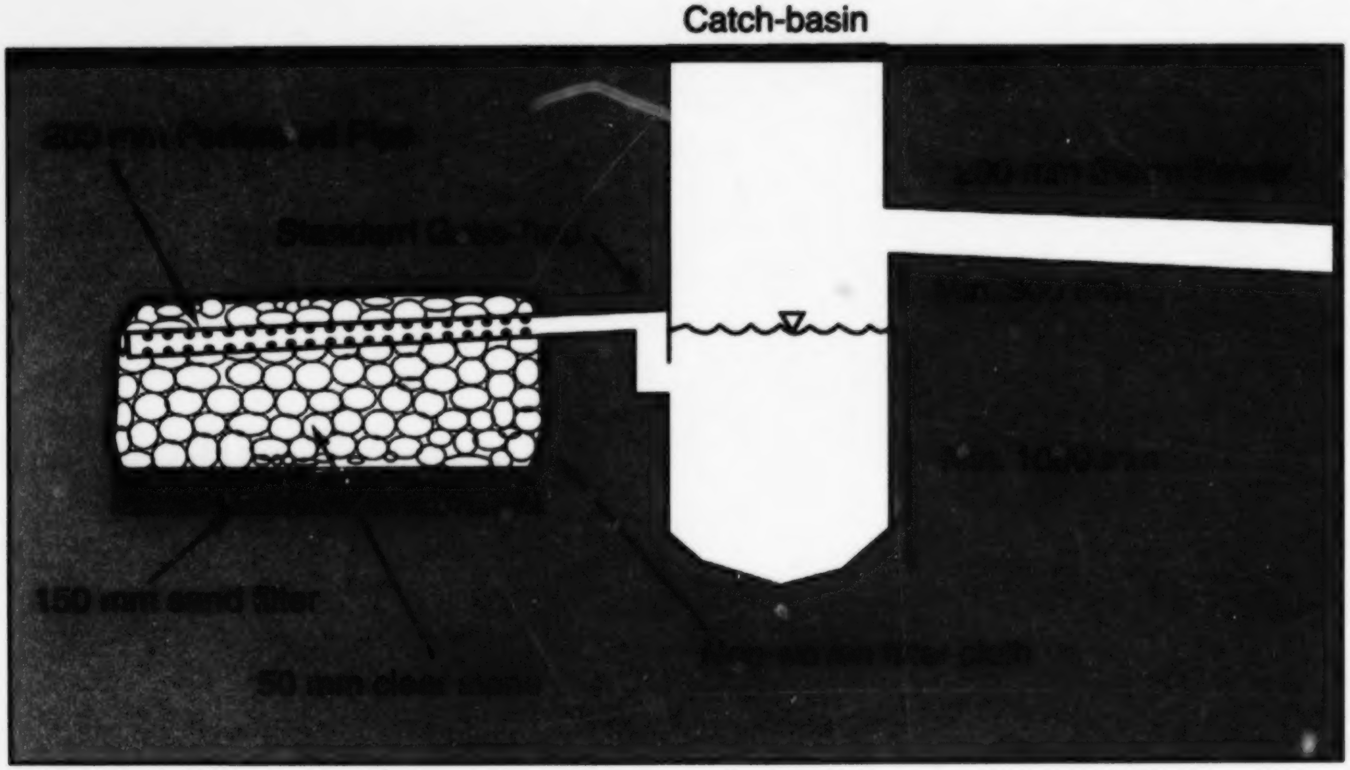
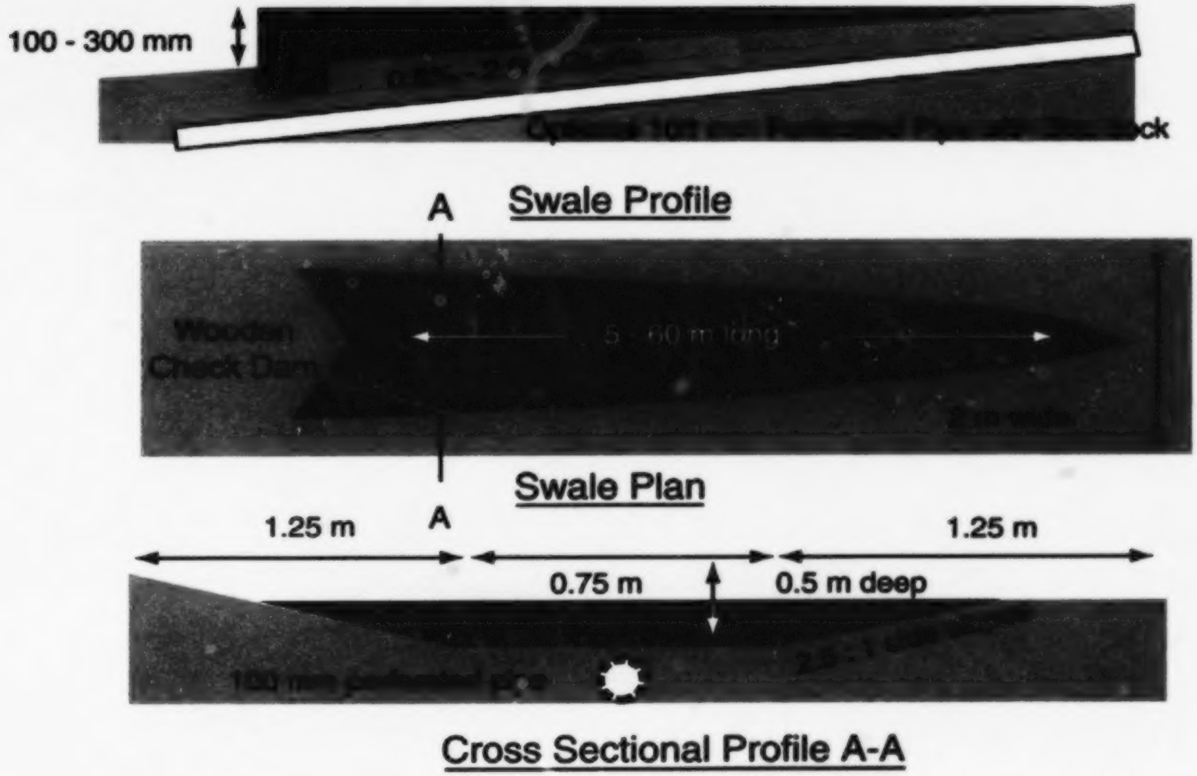

Catch-basin
200 mm Perforated Pipe
Standard Goss Trap
250 mm Storm Sewer
Min. 300 mm
Min. 1000 mm
150 mm sand filter
50 mm clear stone
Non-woven filter cloth

Figure 6.7

Pervious Catch-Basin


100 - 300 mm
0.5% - 2.5%
Optional 100 mm Perforated Pipe and Crushed Rock
A
Wooden Check Dam
5 - 60 m long
2 m wide
A
1.25 m
0.75 m
1.25 m
0.5 m deep
2.5 : 1 side slope
100 mm perforated pipe

Swale Profile

Swale Plan

Cross Sectional Profile A-A

Figure 6.8

Grass Swale Design

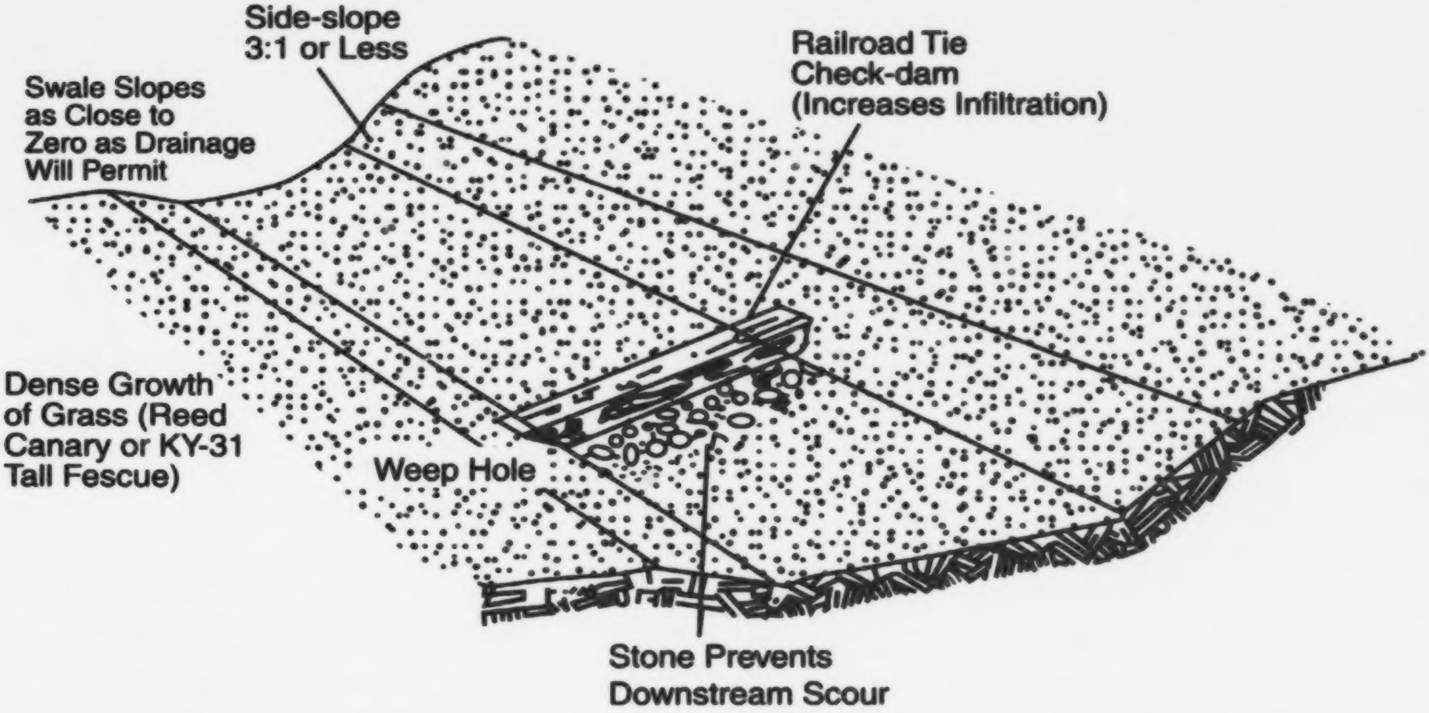

Side-slope
3:1 or Less
Swale Slopes
as Close to
Zero as Drainage
Will Permit
Railroad Tie
Check-dam
(Increases Infiltration)
Dense Growth
of Grass (Reed
Canary or KY-31
Tall Fescue)
Weep Hole
Stone Prevents
Downstream Scour

Figure 6.9

Grass Swale with Check Dam

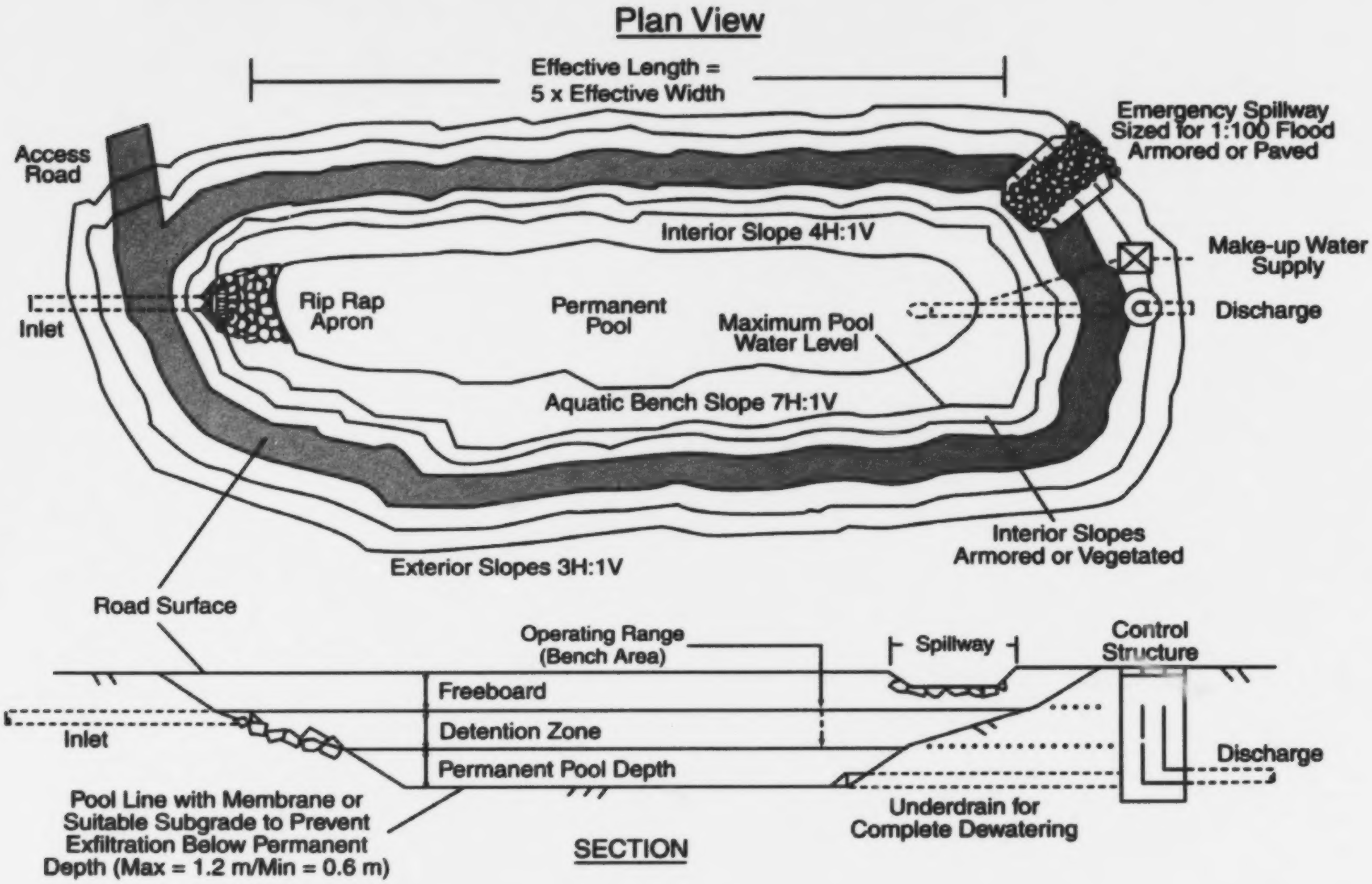

Plan View
Effective Length =
5 x Effective Width
Access Road
Emergency Spillway
Sized for 1:100 Flood
Armored or Paved
Interior Slope 4H:1V
Make-up Water
Supply
Inlet
Rip Rap
Apron
Permanent
Pool
Maximum Pool
Water Level
Discharge
Aquatic Bench Slope 7H:1V
Interior Slopes
Armored or Vegetated
Exterior Slopes 3H:1V
Road Surface
Operating Range
(Bench Area)
Spillway
Control
Structure
Freeboard
Inlet
Detention Zone
Discharge
Permanent Pool Depth
Pool Line with Membrane or
Suitable Subgrade to Prevent
Exfiltration Below Permanent
Depth (Max = 1.2 m/Min = 0.6 m)
Underdrain for
Complete Dewatering
SECTION

Figure 6.10
Wet Detention Pond Plan and Sections

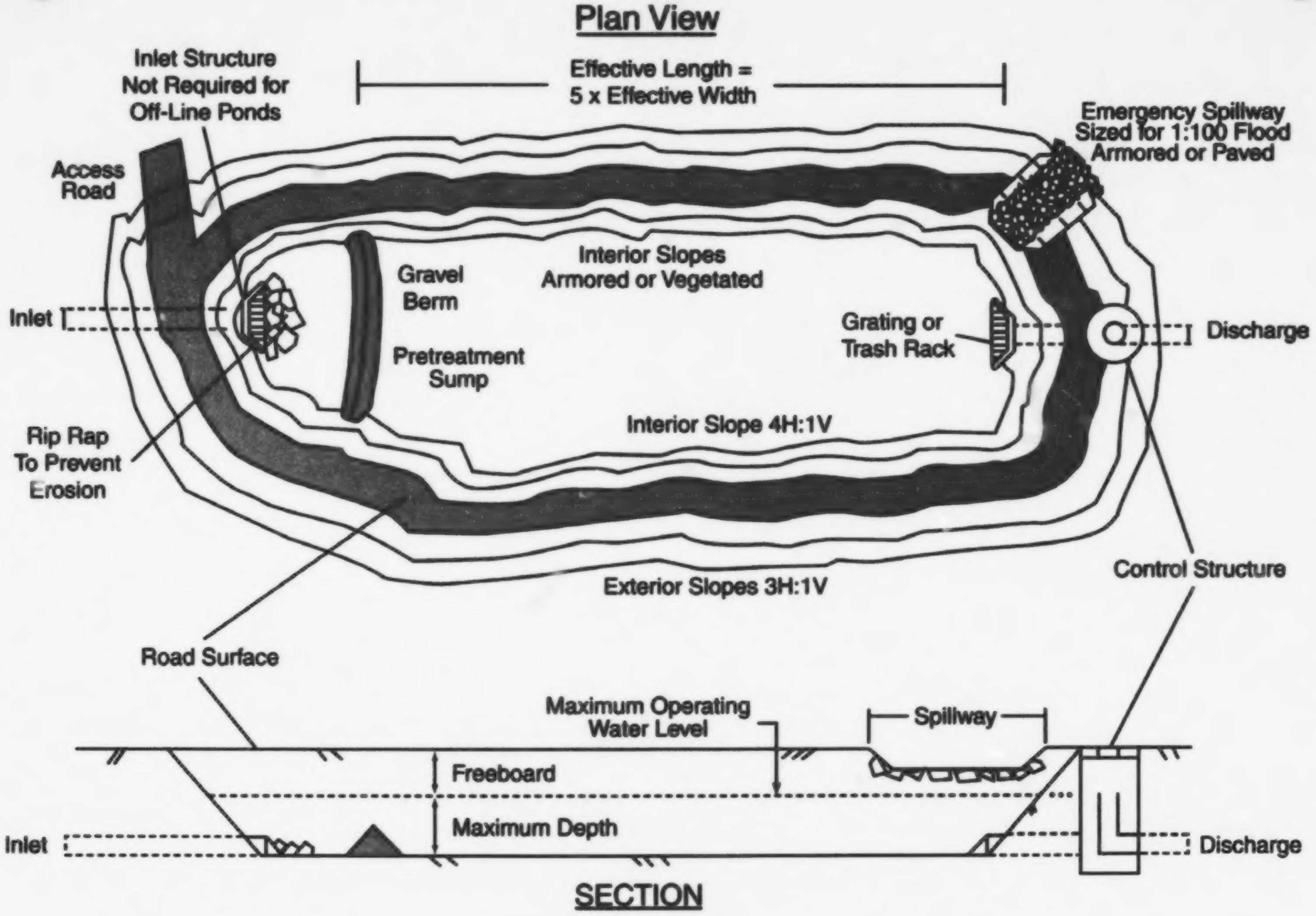
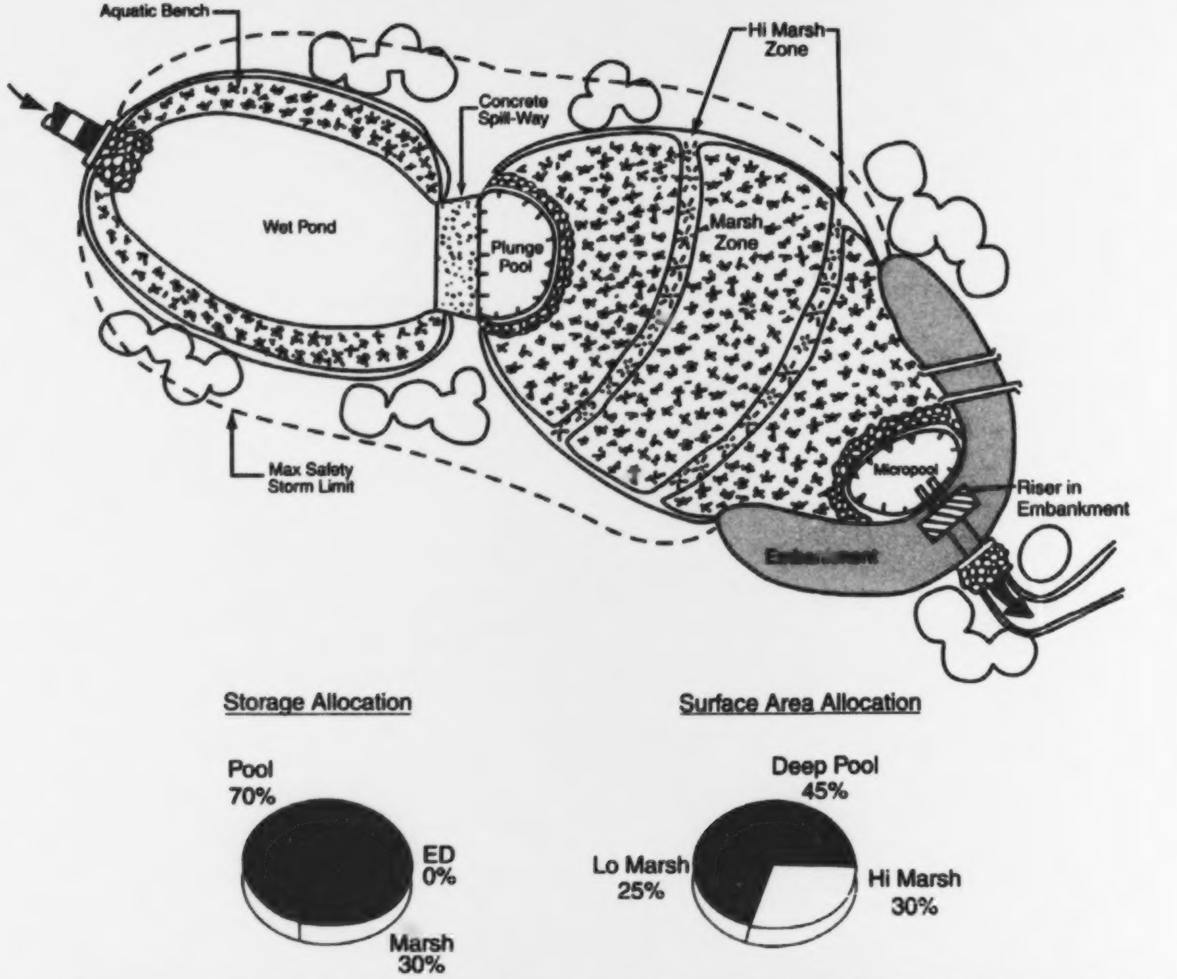

Plan View
Inlet Structure Not Required for Off-Line Ponds
Effective Length = 5 x Effective Width
Emergency Spillway Sized for 1:100 Flood Armored or Paved
Access Road
Gravel Berm
Interior Slopes Armored or Vegetated
Inlet
Grating or Trash Rack
Discharge
Pretreatment Sump
Interior Slope 4H:1V
Rip Rap To Prevent Erosion
Control Structure
Exterior Slopes 3H:1V
Road Surface
Maximum Operating Water Level
Spillway
Freeboard
Maximum Depth
Inlet
Discharge
SECTION

Figure 6.11

Dry Detention Pond Plan and Sections


Aquatic Bench
Hi Marsh Zone
Concrete Spill-Way
Wet Pond
Plunge Pool
Marsh Zone
Micropool
Riser in Embankment
Max Safety Storm Limit
Embankment
Storage Allocation
Pool 70%
ED 0%
Marsh 30%
Surface Area Allocation
Deep Pool 45%
Lo Marsh 25%
Hi Marsh 30%

The pond/wetland system consists of two separate cells - a deep pond leading to a shallow wetland. The pond removes pollutants, and reduces the space required for the system.

Figure 6.12

Stormwater Wetland

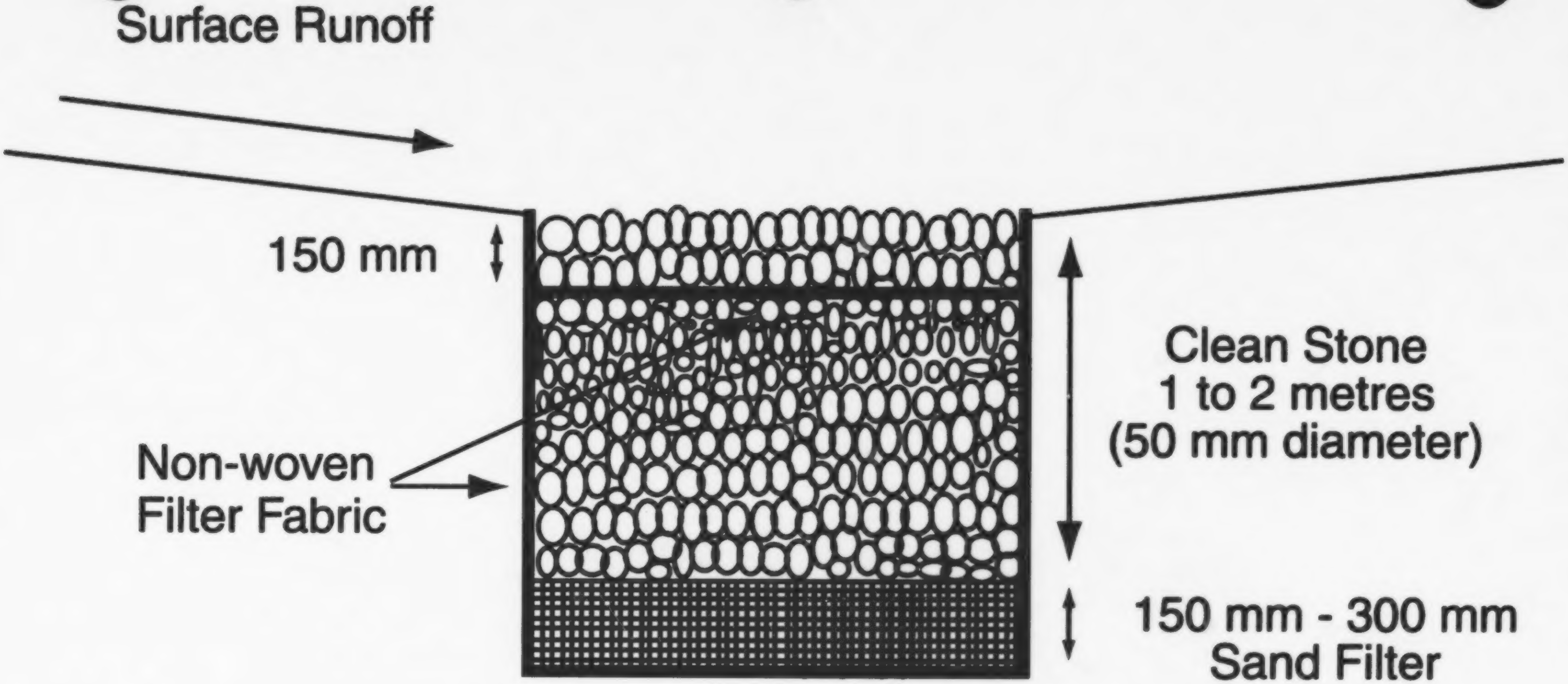

Surface Runoff
150 mm
Clean Stone
1 to 2 metres
(50 mm diameter)
Non-woven
Filter Fabric
150 mm - 300 mm
Sand Filter

Figure 6.13

Surface Infiltration Trench

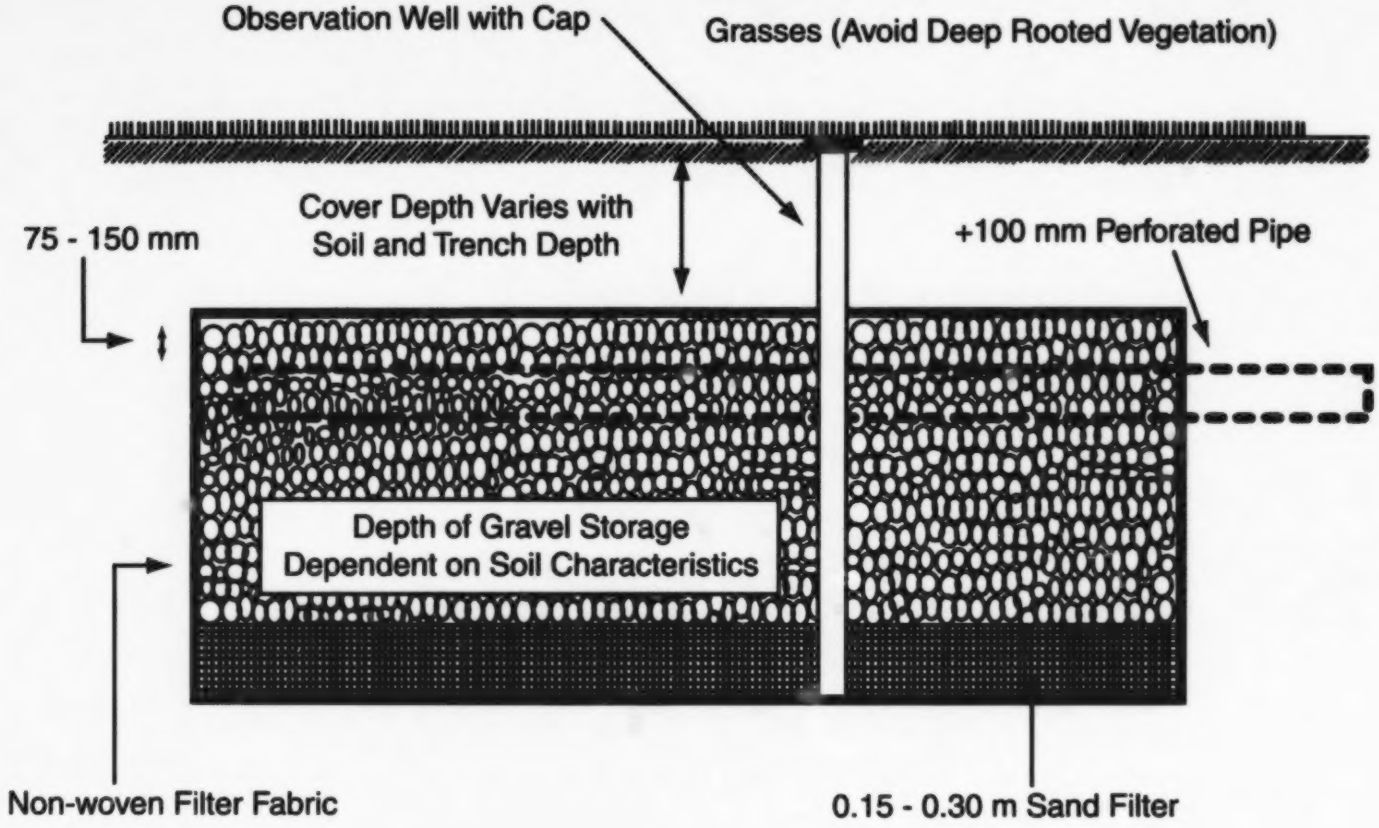

Observation Well with Cap
Grasses (Avoid Deep Rooted Vegetation)
Cover Depth Varies with Soil and Trench Depth
75 - 150 mm
+100 mm Perforated Pipe
Depth of Gravel Storage Dependent on Soil Characteristics
Non-woven Filter Fabric
0.15 - 0.30 m Sand Filter

Figure 6.14

Subsurface Infiltration Trench

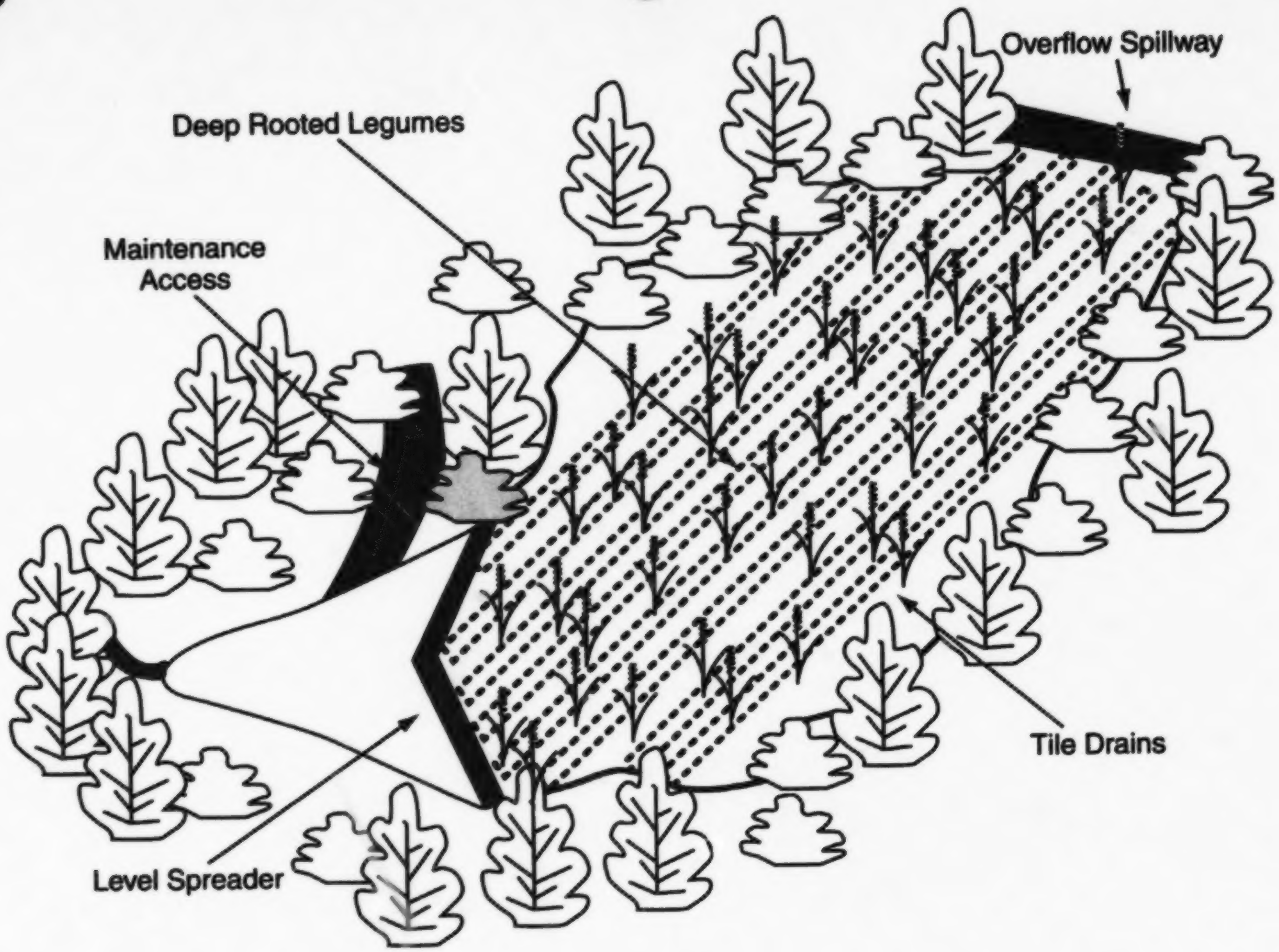

Overflow Spillway
Deep Rooted Legumes
Maintenance Access
Tile Drains
Level Spreader

Figure 6.15

Infiltration Basin

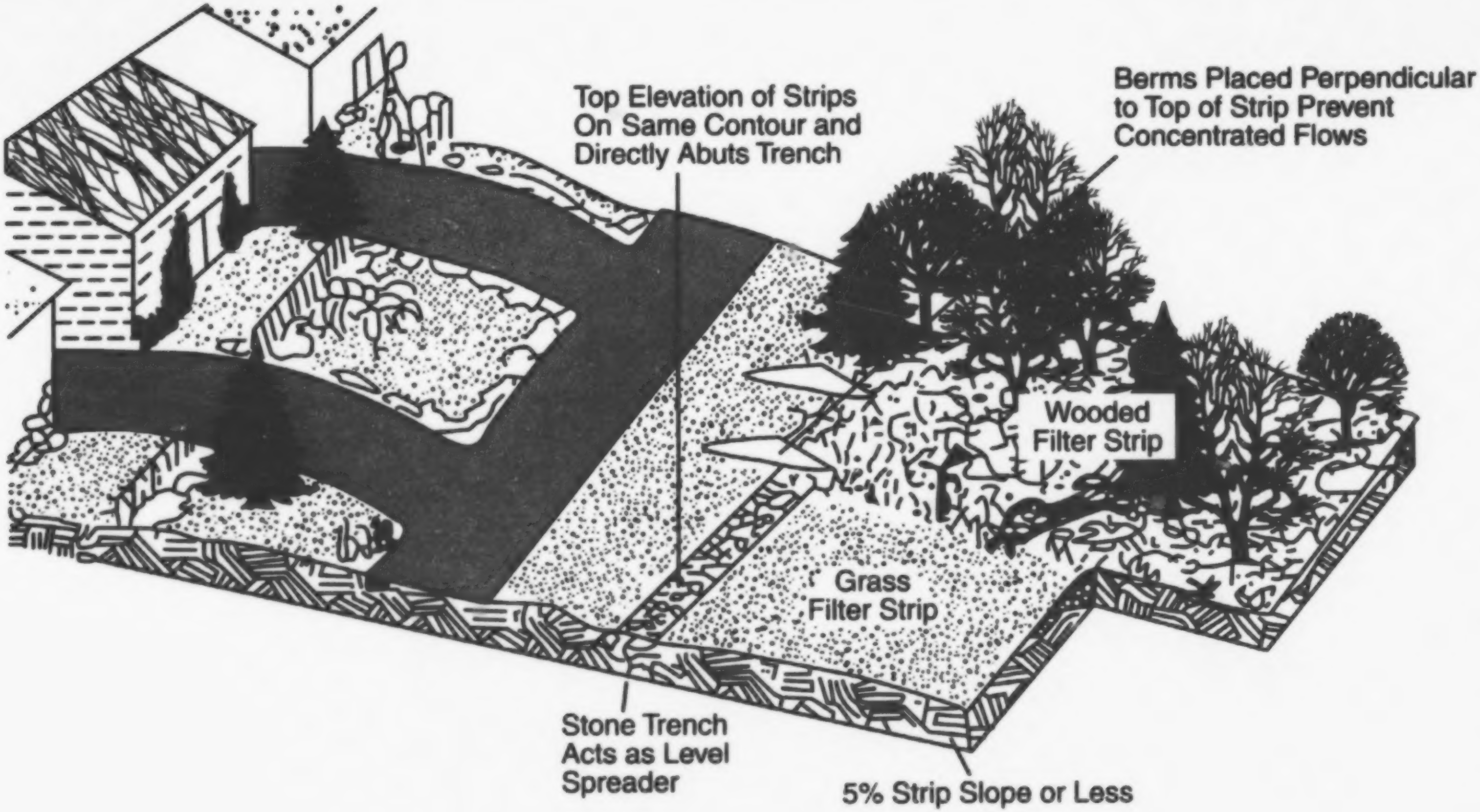

Top Elevation of Strips
On Same Contour and
Directly Abuts Trench
Berms Placed Perpendicular
to Top of Strip Prevent
Concentrated Flows
Wooded
Filter Strip
Grass
Filter Strip
Stone Trench
Acts as Level
Spreader
5% Strip Slope or Less

Figure 6.16

Schematic of Grassed and Wooded Filter Strip

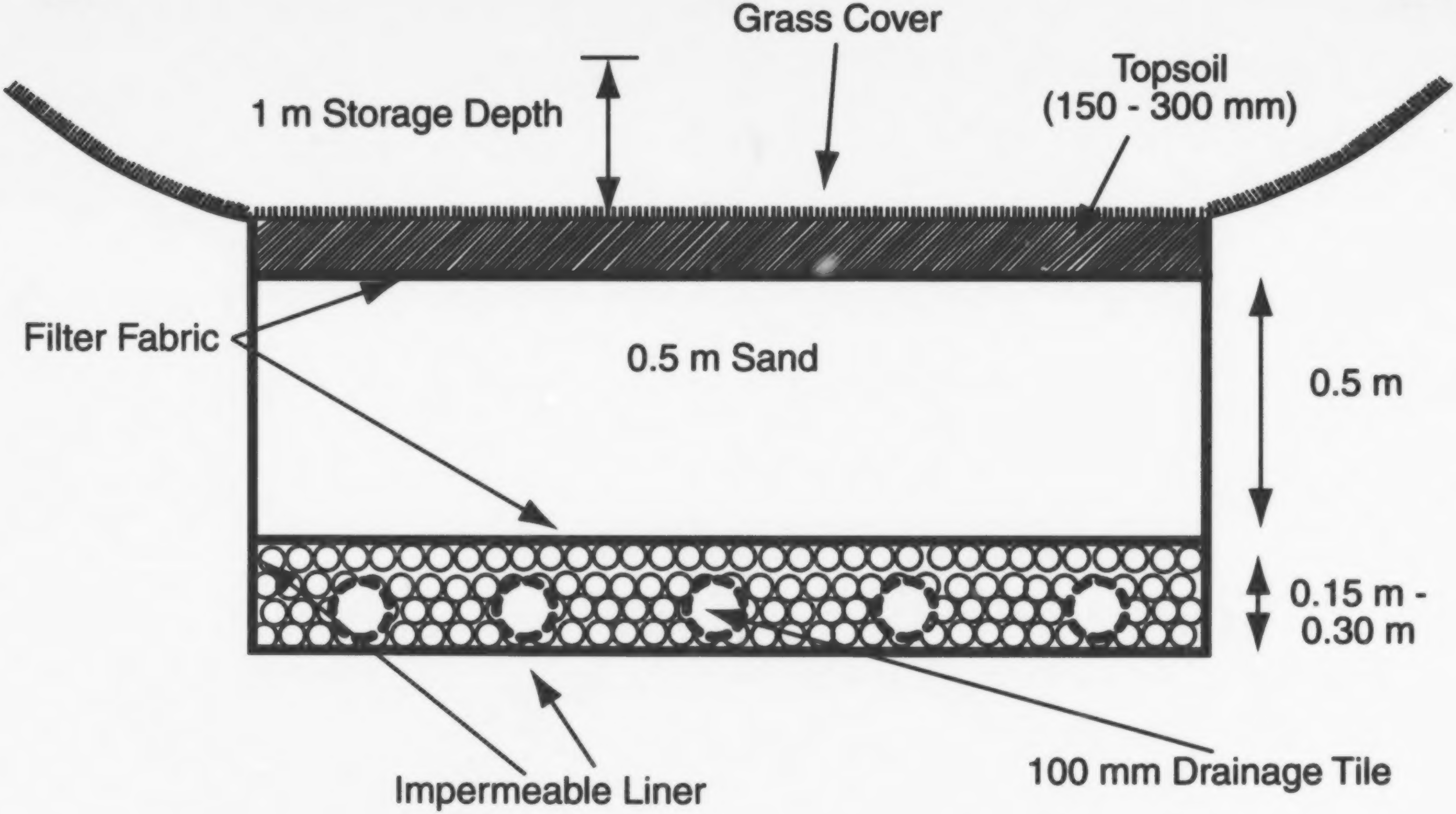

Grass Cover
1 m Storage Depth
Topsoil
(150 - 300 mm)
Filter Fabric
0.5 m Sand
0.5 m
0.15 m -
0.30 m
Impermeable Liner
100 mm Drainage Tile

Figure 6.17

Sand Filter Cross Section Profile

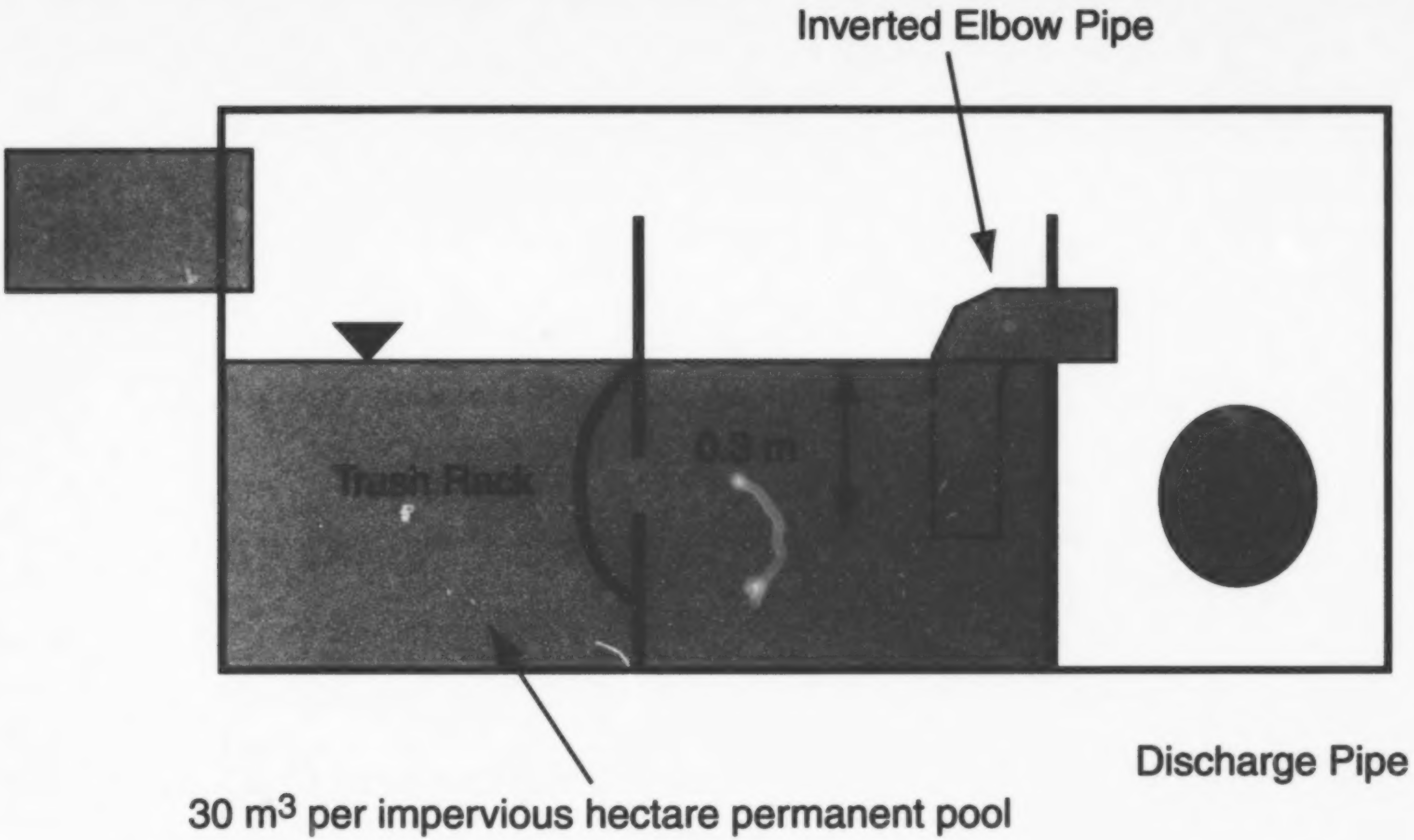

Inverted Elbow Pipe
Trash Rack
0.3 m
Discharge Pipe
30 m^3 per impervious hectare permanent pool

Figure 6.18

Standard 3 Chamber Oil/Grit Separator

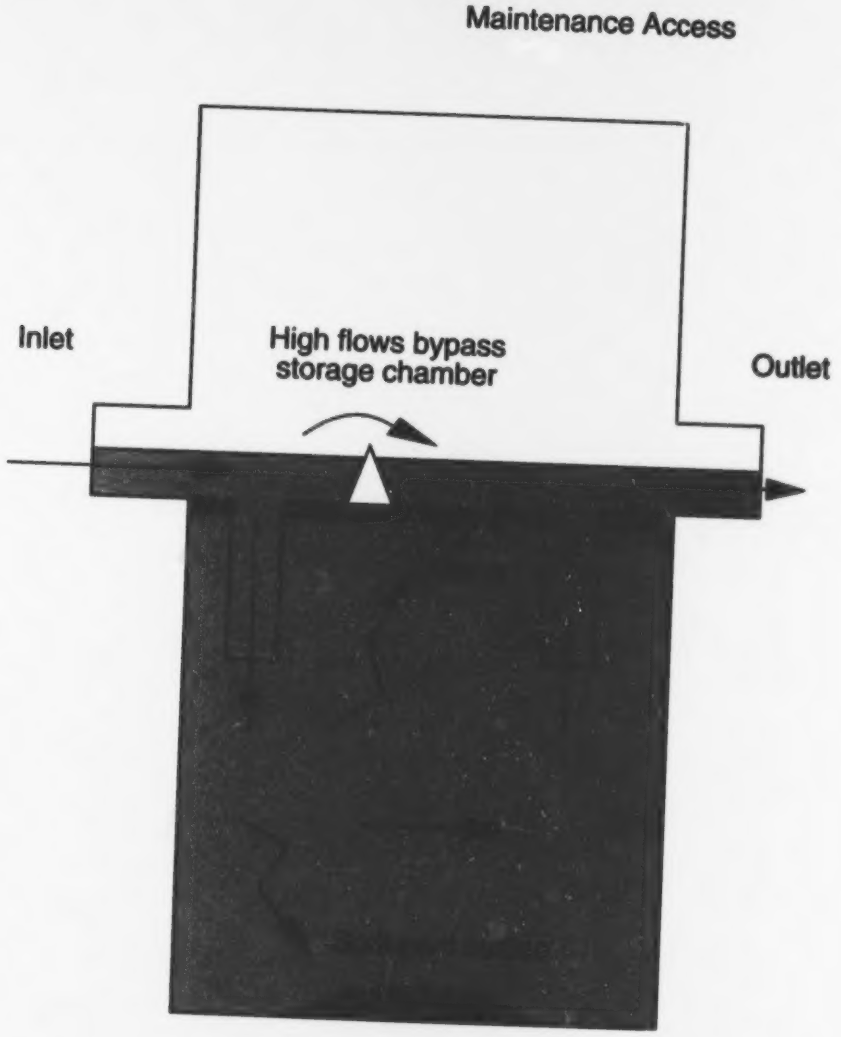

Maintenance Access
Inlet
High flows bypass storage chamber
Outlet

Figure 6.19

Bypass Separator

7.0 Wastewater Systems - Operating and Monitoring (Requirements and Guidelines)

7.1 Systems Operations

7.1.1 General

The proper operation and maintenance of wastewater systems is essential to produce highest quality of treated effluent and to ensure the protection of public health and the environment. It is therefore important that programs and activities such as good operator training, emergency response planning, corrective action measures, etc. are in place to ensure a reliable and well-operated wastewater system.

7.1.2 Reliability

1. The wastewater system should produce effluent to meet the required limits at all times. Consideration should be given to optimize operation of the system to handle both dry weather and wet weather flows.

2. The owner should ensure that the system is operated, maintained, and has appropriate backup facilities to protect against failures of the power supply, treatment process, equipment, or structure.

7.1.3 Operations

1. The wastewater systems should be managed and operated in accordance with the EPEA approval of the systems. The wastewater treatment facilities should be operated to produce effluent that meet the standards detailed in Tables 3.1 and 3.2.

2. Non-domestic discharges should not interfere with the operation of the treatment plant, nor should it impact on the treatability of the wastewater and affect the performance of the plant.

3. The owner should ensure the development and implementation of an emergency response plan as part of the operations program, for emergencies such as pipeline breakage or accidental spills of any toxins to sewers and/or treatment plant. The plan should include:

    i) General procedures for routine or major emergencies within the wastewater system; and

    ii) A contingency plan for facilities becoming inoperable in a major emergency.

4. The plant should be operated within its design capacity.

5. The owner should take preventative or corrective action as directed by AENV when results of an inspection conducted by AENV or monthly returns indicate conditions which are currently or may become a detriment to system operations.

7.1.4 Facility Classification And Operator Requirements

7.1.4.1 Facility Classification

On recommendation from Water and Wastewater Operators Certification Advisory Committee, AENV will classify all wastewater facilities. Facility classification may also be reviewed upon request by the owner or authorized representative. The classification of Wastewater Collection (WWC) system is based upon the population served by the facilities while the classification of Wastewater Treatment (WWT) facilities is based upon a range of points as shown in Tables 7.1(a).

Table 7.2 summarizes the classification system. The classification system is based on the "degree of difficulty to operate" a facility. The Alberta system is similar to many used across Canada and the United States.

7.1.4.2 Requirement For Having Certified Operators

In accordance with EPEA, day-to-day operations of wastewater systems should be supervised by one or more persons who hold a valid certificate of qualification for the type of class of facility concerned. The Approval for each facility will state the required number of certified operators and their required level of certification.

Exempted from these requirement are:

1. "privately owned developments" as defined in the regulations; and
2. any other systems as determined by AENV.

7.1.4.3 Responsibility Of Operators

It is the responsibility of certified operators to know and understand the terms and conditions in the operating Approval for their facility. It is also their responsibility to understand the certification requirements for operators of their facilities as indicated by the Approval or by the Certification Guidelines.

It is necessary that the chief operator ensure current certification for operators as required by the Approval or by the Certification Guidelines. It is also important that each facility has a contingency plan so that certified operator requirements are met in cases of planned absences (e.g. vacation), unplanned absences (e.g. illness), or change of staff (e.g. retirement).

Certified operators are also responsible to establish or understand contingency plans for each facility that ensure that the Approval requirements, with respect to certified operators, are met at all times. This is important during normal operation or in the cases of planned absences (e.g. vacation), unplanned absences (e.g. illness), or change of staff (e.g. retirement).

7.1.4.4 Responsibility of Facility Owners

It is the legal responsibility of the owner of each facility to be aware of the requirements of the Approval and to ensure that the requirements are met. The Approval issued by AENV will designate the minimum number and level of certification of key operations personnel. It is important that facility owners develop an internal program so that substitute or replacement personnel are available when necessary.

TABLE 7.1(a)
CLASSIFICATION OF WASTEWATER TREATMENT PLANTS (WWT)

ITEM	POINTS
Size	
Maximum population equivalent (P.E.) served, peak day (1 pt/10,000 P.E. or part)	0 - 5
Design flow (avg. day) or peak month's flow (avg. day), whichever is larger (1 pt/5,000 m^3/day)	0 - 5
Effluent Discharge	
Receiving stream (sensitivity)*	0 - 6*
Land disposal - evaporation	2
Subsurface disposal	4
Variation in Raw Wastewater (slight to extreme)*	0 - 6*
Pre-treatment	
Plant pumping of main flow	3
Screening, comminution	3
Grit removal	3
Chemical pre-treatment except chlorination, enzymes	4
Primary Treatment	
Primary clarifiers	5
Combined Sedimentation/digestion	5
Secondary Treatment	
Trickling filter w/sec. clarifiers or RBC	10
Activated sludge w/sec. clarifiers (including ext. aeration and oxidation ditches)	15
Stabilization ponds without aeration	5
Aerated lagoon	8
Advanced Waste Treatment	
Polishing pond	2
Chemical/physical - without secondary	15
Chemical/physical - following secondary	10
Biological or chemical/biological	12
Ion exchange	10
Reverse osmosis, electrodialysis	15
Chemical recovery, carbon regeneration	4
Solids Handling	
Conditioning	2
Thickening	5
Anaerobic digestion	10
Aerobic digestion	6
Evaporative sludge drying	2
Mechanical dewatering	8
Solids reduction (incineration, wet oxidation)	12
Disinfection	
Chlorination or comparable	5
On-site generation	5
Laboratory Control by Plant Personnel	
Bacteriological (complexity)*	0 - 10*
Chemical/physical (complexity)*	0 - 10*

Note: Each unit process should have points assigned only once.
* See Table 7.1(b)

TABLE 7.1(b)
WASTEWATER TREATMENT PLANT CLASSIFICATION POINT GUIDE

ITEM	POINTS
Receiving Stream Sensitivity	0 - 6
The key factor is the degree of dilution provided under low flow conditions. Suggested point values are:	
No discharge	0
Secondary treatment is adequate.	1
Tertiary treatment is required	2
Stream conditions are very critical (run dry, for example)	3
Effluent used in a direct recycle and reuse system	6
Variation in Raw Wastewater Quality	
The key factor is the frequency and/or the extent of variation from normal or typical fluctuations; such deviation can be in terms of strength, toxicity, and shock loads, etc. Suggested point values are:	
Variations do not exceed those normally or typically expected	0
Recurring deviations or excessive variations of 100 to 200 percent in strength and/or flow	2
Recurring deviations or excessive variations of more than 200 percent in strength and/or flow	4
Raw wastes subject to toxic waste discharges	6
Laboratory Control By Plant Personnel	
Bacteriological/biological (complexity)	0 - 10
The key concept is to credit bacti/bio lab work done on-site by plant personnel. Suggested point values are:	
Lab work done outside the plant	0
Membrane filter procedures	3
Use of fermentation tubes or any dilution methods; fecal coliform determination	5
Biological identification	7
Virus/parasite studies or similarly complex work conducted on-site	10
Chemical/physical (complexity)	0 - 10
The key concept is to credit chemical/physical lab work done on-site by plant personnel. Suggested point values are:	
Lab work done outside the plant	0
Push button or visual methods for simple test such as pH, settleable solids - up to	3
Additional procedures such as DO, COD, BOD, solids, gas analysis, titrations, volatile content - up to	5
More advanced determinations such as specific constituents: nutrients, total oils, phenols, etc.	7
Highly sophisticated instrumentation such as atomic absorption and gas chromatography	10

TABLE 7.2
FACILITY CLASSIFICATION SYSTEM

FACILITY	BASED UPON	I	II	III	IV
WWC*	Population Served	1500 or fewer	1501-15,000	15,001-50,000	50,001 or more
WWT	Range of Points (Table 7.1(a))	30 or fewer	31-55	56-75	76 or more

Notes: AENV may adjust the classification of a facility if the point system does not reflect the actual complexity of that facility.

WWC - Wastewater Collection

WWT - Wastewater Treatment

Wastewater pumping and transmission may be part of either wastewater treatment or wastewater collection but, alone, it is not considered to be either wastewater treatment or wastewater collection.

*Simple "in-line" treatment (such as odour control) is considered an integral part of the collection system.

7.1.4.5 Facilities Staffing Requirements: Certified Operators

For Class I facilities, there must be a certified Small Systems or Level I (or higher) operator in charge of the day to day operation of that facility. A back up certified operator is **recommended**.

For Class II facilities, there must be a certified Level II (or higher) operator in charge of the day-to-day operation of that facility. It is recommended that an assistant operator with Level I or II certification be available.

For Class III facilities serving a population under 1,500, there must be a certified Level III (or higher) operator in charge of the day-to-day operation.

For Class III facilities serving a population of 1,500 - 10,000, there must be a certified Level III (or higher) operator in charge of the day-to-day operation. There must also be at least one other operator certified at Level I or higher.

For Class III facilities serving a population of 10,000 - 50,000, there must be a certified Level III (or higher) operator in charge of the day-to-day operation. There must also be at least one other operator certified at Level II or higher.

For Class III facilities serving a population over 50,000, there must be a certified Level III (or higher) operator in charge of the day-to-day operation. There must also be another operator certified at Level II or higher to act in the absence of the charge operator and at least one other operator certified at Level I or higher. There must be at least one certified operator (any level) for each shift when shift operation is required.

For Class IV facilities serving a population up to 200,000, there must be a Level IV operator in charge of the day-to-day operation. There must also be two Level III (or higher) operators to act in the absence of the Level IV operator. There must be at least one Level II or higher certified operator for each shift when shift operation is required.

For Class IV facilities serving a population over 200,000, there must be a certified Level IV operator in charge of the day-to-day operation. There must be at least one other certified Level IV operator to act in the absence of the charge operator. There must be a third operator who is certified at either Level III or IV and there must be at least one Level II (or higher) certified operator for each shift when shift operation is required.

7.2 System Monitoring

7.2.1 General

Alberta Environment considers monitoring to fall into one of the following categories:

1. operational monitoring
2. treatment performance and compliance monitoring
3. issue oriented and follow-up monitoring.

Types of monitoring are discussed in detail in the next few sections.

7.2.1.1 Sampling Procedures and Analytical Methods

1. The usefulness of any monitoring program depends to a large extent on the sampling procedures used. It is important to ensure that the sample collected is truly representative of the wastewater stream.

2. Based on the type of analysis, AENV would require two types of samples to be collected, "composite" or "grab."

   "Composite sample" means a sample consisting of not less that twenty-four discrete portions of equal volume collected as follows:

   i) at time intervals directly proportional to the flow rate of the liquid being sampled during each time interval, with a minimum of one discrete sample collected every hour over a period of 24 hours, or

   ii) sequentially at regular time intervals over a period of 24 hours.

   "Grab sample" means an individual sample collected in less than 15 minutes and which is representative of the wastewater being sampled.

3. The owner should ensure that:

   i) Collection and preservation of samples and all analytical procedures are:

      - in accordance with the latest edition of "Standard Methods for the Examination of Water and Wastewater," as published by the American Public Health Association, American Water Works Association, and the Water Pollution Control Federation; or

      - by a method outlined in the most recent edition of the "Methods Manual for Chemical Analysis of Water and Wastes" or "Methods Manual for Chemical Analysis of Trace Organics and Pesticides in Environmental Samples," published by Alberta Environmental Protection; or

      - by an alternative method approved by AENV; or

   ii) Collection, preservation and the analysis of samples are performed by a laboratory approved by AENV.

7.2.1.2 Approval of Analytical Procedures

Owners should ensure that laboratories obtain approval from AENV for the use of any analytical procedures not included in the "Standard methods."

The laboratory would be required to follow a protocol established by AENV for the approval of analytical procedures not included in the "Standard Methods."

7.2.2 Operational Monitoring

The extent and the complexity of operational monitoring is dependent on the size and the type of the facility. For instance, there are a number of operating variables which are vitally important to the proper functioning of an activated sludge system. Some of these are under the Operator's control, and some are not. Table 7.3 lists some of the significant parameters that should be monitored to ensure proper operation of an activated sludge system. Operational monitoring requirements are established both for specific process control purposes, and to ensure that a facility receives good operational attention on a regular basis.

Table 7.4 outlines the operational monitoring requirements for wastewater stabilization ponds.

7.2.2.1 Activated Sludge Systems

TABLE 7.3
OPERATIONAL MONITORING

Parameters	Point of Measurement	Requirement	Type of Sample	Monitoring Frequency
Flow (raw)	Headworks	Not to exceed design capacity	N/A	Daily from totalizer
Flow (treated)	Prior to Outfall	N/A	N/A	Daily from totalizer
Peak flow (raw)	Headworks	N/A	N/A	Daily from strip - chart or PLC
TSS (raw)	Headworks	N/A	Composite	Daily
TSS (primary)	Downstream of primary clarifier	Varies; required to determine the aeration capacity	Composite	Daily
TSS (RAS)	RAS line	Varies; required to control solids in aeration tank	Grab/Composite	Daily
TSS (treated)	Prior to outfall	Refer to compliance limit	Composite	Daily
MLSS	Aeration tank	800-2000 mg/L (without nitrification)	Grab	Daily
		2000-4000 mg/L (with nitrification)	Grab	Daily
CBOD (raw)	Headworks	N/A	Composite	Daily
COD (primary)	Downstream of primary clarifier	Varies; required to determine the aeration capacity	Composite	Daily
CBOD (treated)	Prior to outfall	Refer to compliance limit	Composite	Daily
Sludge Volume Index	Aeration tank/calculated	< 150 mL/g	Grab	Daily
Sludge Age	Calculated	3-10 days (without nitrification)	N/A	Daily
		10-15 days (with nitrification)	N/A	Daily
F:M ratio	Calculated	0.05 - 0.5	N/A	Twice Weekly
Dissolved Oxygen	Aeration tank	2 mg/L	N/A	Continuous

7.2.2.2 Wastewater Lagoons

TABLE 7.4
OPERATIONAL MONITORING

Requirements	Monitoring Frequency
Actual dates and duration of discharge	Annually for every discharge
Volume of discharge	Annually for every discharge
Monitoring of each groundwater observation well for: water level, TKN NH_3-N, NO_3-N, NO_2-N, TDS, COD, and any other parameter as determined by AENV.	For new lagoons, four times in each quarter of the first year of operation. The first analysis from each well prior to putting the new lagoon into operation. The following three analyses approximately three months apart to cover all seasons of the year.
Monitoring of each groundwater observation well for water level during the discharge period.	One set of readings immediately before discharge, one set immediately after discharge, and one set approximately one month after the end of the discharge period.

7.2.3 Treatment Performance and Compliance Monitoring

Treatment performance and compliance monitoring will be dependent on a number of factors, including:

- type of treatment (mechanical secondary treatment plants, mechanical tertiary treatment plants, aerated lagoons, wastewater stabilization ponds)
- type of discharge (continuous, intermittent)
- type of receiving body (water or land).

The Wastewater and Storm Drainage Regulation and the Wastewater and Storm Drainage (Ministerial) Regulation (119/93 and 120/93) require physical, microbiological, radiological or chemical analyses of wastewater and storm drainage samples for those parameters specified by AENV. This section outlines the specific parameters that have to be monitored, including the sample location and the monitoring frequency, for different types of treatment method, discharge and receiving body.

7.2.3.1 Secondary Treatment - Mechanical (for current population < 20,000)

1. Continuous Discharge to a body of water

TABLE 7.5
TREATMENT PERFORMANCE AND COMPLIANCE MONITORING

Parameter	Point of Measurement	Requirement	Type of Sample	Minimum Monitoring Frequency
CBOD	Prior to outfall	Monthly averaging of daily samples shall not exceed 25 mg/L	Composite	Daily
TSS	Prior to outfall	Monthly average of daily samples shall not exceed 25 mg/L	Composite	Daily

2. Intermittent Discharge to a Body of Water

TABLE 7.6
TREATMENT PERFORMANCE AT COMPLIANCE MONITORING

Parameter	Point of Measurement	Requirement	Type of Sample	Minimum Monitoring Frequency
CBOD	Prior to storage cell	Monthly average of three times per week; samples shall not exceed 25 mg/L	Composite	Three times per week during the period of discharge to storage cell
CBOD	Prior to outfall	None	Grab	Once, on the second day of the discharge period. Discharge period **should** not exceed three weeks.
TSS	Prior to storage cell	Monthly average of five times per week samples shall not exceed 25 mg/L	Composite	Five times per week during the period of discharge to storage cells, excluding statutory holidays.
TSS	Prior to outfall	None	Grab	Once, on the second day of the discharge period. Discharge period should not exceed three weeks, unless local conditions preclude this rate of discharge.

3. Continuous and/or intermittent discharge to land (Effluent Irrigation)

TABLE 7.7
TREATMENT PERFORMANCE AND COMPLIANCE MONITORING

Parameter	Point of Measurement	Requirement	Type of Sample	Minimum Monitoring Frequency
CBOD	Prior to irrigation	< 100 mg/L	Grab/composite*	Twice annually, prior to and on completion of a major application event
TSS	Prior to irrigation	< 100 mg/L	Grab/composite*	Twice annually, prior to and on completion of a major application event
EC	Prior to irrigation	< 2.5 mmhos/cm	Grab/composite*	Twice annually, prior to and on completion of a major application event
SAR	Prior to irrigation	< 9	Grab/composite*	Twice annually, prior to and on completion of a major application event
pH	Prior to irrigation	6.5 - 9.5	Grab/composite*	Twice annually, prior to and on completion of a major application event
Total Coliform	Prior to irrigation (golf course/parks only)	Geometric mean of weekly samples (if storage provided) and daily samples (if storage not provided), in a 30-day period shall not exceed 1000/100 mL	Grab	Daily**/weekly during the irrigation season
Fecal Coliform	Prior to irrigation (golf course/parks only)	Geometric mean of weekly samples (if storage provided) and daily samples (if storage not provided), in a 30-day period shall not exceed 200/100 mL	Grab	Daily**/weekly during the irrigation season

* Grab sample if storage provided; composite sample if storage not provided

** Frequency of sampling will be reduced if it can be demonstrated with some certainty that bacteriological quality of effluent is consistent and the probability of variance is minimal.

7.2.3.2 Tertiary Treatment - Mechanical (for current population > 20,000)

1. **Continuous discharge to a body of water**

TABLE 7.8
TREATMENT PERFORMANCE AND COMPLIANCE

Parameter	Point of Measurement	Requirement	Type of Sample	Minimum Monitoring Frequency
CBOD	Prior to outfall	Monthly average of daily samples shall not exceed 20 mg/L	Composite	Daily
TSS	Prior to outfall	Monthly average of daily samples shall not exceed 20 mg/L	Composite	Daily
TP	Prior to outfall	Monthly average of daily samples shall not exceed 1 mg/L	Composite	Daily
NH_3-N	Prior to outfall	Assessed on a site specific basis	Composite	Assessed on a site specific basis
Total Coliform	Prior to outfall	Geometric mean of daily samples in a 30 day period shall not exceed 1000/100 mL	Grab	*Daily
Fecal Coliform	Prior to outfall	Geometric mean of daily samples in a 30 day period shall not exceed 200/100 mL	Grab	*Daily

* Frequency of sampling will be reduced if it can be demonstrated with some certainty that bacteriological quality of effluent is consistent and the probability of variance is minimal.

2. Intermittent Discharge to a Body of Water

TABLE 7.9
TREATMENT PERFORMANCE AND COMPLIANCE MONITORING

Parameter	**Point of Measurement**	**Requirement**	**Type of Sample**	**Minimum Monitoring Frequency**
CBOD	Prior to storage cell	Monthly average of three times per week samples shall not exceed 20 mg/L	Composite	Three times per week during the period of discharge to storage cell
CBOD	Prior to outfall	None	Grab	Once, on the second day of the discharge period. Discharge period shall not exceed three weeks, unless local conditions preclude this rate of discharge.
TSS	Prior to storage cell	Monthly average of five times per week samples shall not exceed 20 mg/L	Composite	Five times per week during the period of discharge to storage cells, excluding statutory holidays.
SS	Prior to outfall	None	Grab	Once, on the second day of the discharge period. Discharge period should not exceed three weeks unless local conditions preclude this rate of discharge.
TP	Prior to storage cell	Monthly average of five times per week samples shall not exceed 1 mg/L	Composite	Five times per week during the period of discharge to storage cells, excluding statutory holidays

Table 7.9 - Continued

Parameter	Point of Measurement	Requirement	Type of Sample	Minimum Monitoring Frequency
TP	Prior to outfall	None	Grab	Once, on the second day of the discharge period. Discharge period should not exceed three weeks, unless local conditions preclude this rate of discharge.
NH_3-N	Prior to outfall	Assessed on a site specific basis	Grab	Assessed on a site-specific basis.
Total Coliform	Prior to storage cell	Geometric mean of three times per week; samples in a calendar month shall not exceed 1000/100 mL	Grab	*Three times per week during the period of discharge to storage cell.
Total Coliform	Prior to outfall	None	Grab	Once, on the second day of the discharge period. Discharge period should not exceed three weeks, unless local conditions preclude this rate of discharge.
Fecal Coliform	Prior to storage cell	Geometric mean of three times per week samples in a calendar month shall not exceed 200/100 mL	Grab	*Three times per week during the period of discharge to storage cell.
Fecal Coliform	Prior to outfall	None	Grab	Once, on the second day of the discharge period. Discharge period should not exceed three weeks, unless local conditions preclude this rate of discharge.

* Frequency of sampling will be reduced if it can be demonstrated with some certainty that bacteriological quality is consistent and the probability of variance is minimal.

3. Continuous and/or intermittent discharge to land (Effluent Irrigation)

Tertiary (mechanical) treatment performance and compliance monitoring for effluent irrigation is the same as for secondary (mechanical) treatment monitoring for effluent irrigation. Please refer to Table 7.7.

7.2.3.3 Aerated Lagoons (for current population < 20,000)

1. **Continuous discharge to a body of water**

TABLE 7.10
TREATMENT PERFORMANCE AND COMPLIANCE MONITORING

Parameter	Point of Measurement	Requirement	Type of Sample	Minimum Monitoring Frequency
CBOD	Prior to outfall	Monthly averaging of weekly samples shall not exceed 25 mg/L	Grab	Weekly

2. **Intermittent discharges to a body of water**

TABLE 7.11
TREATMENT PERFORMANCE AND COMPLIANCE MONITORING

Parameter	Point of Measurement	Requirement	Type of Sample	Minimum Monitoring Frequency
CBOD	Prior to storage cell	Monthly average of weekly samples shall not exceed 25 mg/L	Grab	Weekly during the period of discharge to storage cell
CBOD	Prior to outfall	None	Grab	Once, on the second day of the discharge period. Discharge period should not exceed three weeks, unless local conditions preclude this rate of discharge.

3. **Continuous and/or intermittent discharge to land (Effluent Irrigation)**

Aerated lagoon performance and compliance monitoring for effluent irrigation is the same as for secondary (mechanical) treatment monitoring for effluent irrigation. Please refer to Table 7.7.

7.2.3.4 Aerated Lagoons (for current population > 20,000)

1. Continuous discharge to a body of water

TABLE 7.12
TREATMENT PERFORMANCE AND COMPLIANCE MONITORING

Parameter	Point of Measurement	Requirement	Type of Sample	Minimum Monitoring Frequency
CBOD	Prior to outfall	Monthly average of weekly samples shall not exceed 20 mg/L	Grab	Weekly
TP	Prior to outfall	Monthly average of weekly samples shall not exceed 1 mg/L	Grab	Weekly
NH_3-N	Prior to outfall	Assessed on site specific basis	Grab	Assessed on site-specific basis.
Total Coliform	Prior to outfall	Geometric mean of weekly samples in a calendar month shall not exceed 1000/100 mL	Grab	Weekly
Fecal Coliform	Prior to outfall	Geometric mean of weekly samples in a calendar month shall not exceed 200/100 mL	Grab	Weekly

2. **Intermittent discharge to a body of water**

TABLE 7.13
TREATMENT PERFORMANCE AND COMPLIANCE MONITORING

Parameter	Point of Measurement	Requirement	Type of Sample	Minimum Monitoring Frequency
CBOD	Prior to storage cell	Monthly average of weekly samples shall not exceed 20 mg/L	Grab	Weekly during the period of discharge to storage cell
CBOD	Prior to outfall	None	Grab	Once, on the second day of the discharge period. Discharge period should not exceed three weeks, unless local conditions preclude this rate of discharge.
TP	Prior to storage cell	Monthly average of weekly samples shall not exceed 1 mg/L	Grab	Weekly during the period of discharge to storage cell.
TP	Prior to outfall	None	Grab	Once, on the second day of the discharge period. Discharge period should not exceed three weeks, unless local conditions preclude this rate of discharge.
NH_3-N	Prior to outfall	Assessed on a site specific basis	Grab	Assessed on a site specific basis
Total Coliform	Prior to storage cell	Geometric mean of weekly samples in a calendar month shall not exceed 1000/100 ml	Grab	Weekly during the period of discharge to storage cell
Total Coliform	Prior to outfall	None	Grab	Once, on the second day of the discharge period. Discharge period should not exceed three weeks, unless local conditions preclude this rate of discharge.
Fecal Coliform	Prior to storage cell	Geometric mean of weekly samples in a calendar month shall not exceed 200/100 ml	Grab	Weekly during the period of discharge to storage cell.
Fecal Coliform	Prior to outfall	None	Grab	Once, on the second day of the discharge period. Discharge period should not exceed three weeks, unless local conditions preclude this rate of discharge.

7.2.3.5 Wastewater Lagoons

TABLE 7.14
TREATMENT PERFORMANCE AND COMPLIANCE MONITORING

Type	Minimum Requirements
Wastewater stabilization pond built to the specified design configuration as per section 4.2.1.2.	Effluent quality standard not specified Discharge from the storage cell once a year between late spring and fall. Early spring discharged allowed under exceptional circumstances Discharge period should not exceed three weeks, unless local conditions preclude this rate of discharge.

7.2.4 Issue Oriented and Follow-up Monitoring

1. Follow-up action by the owner may be required when the system does not meet or produce effluent to meet the standards stipulated in Section 3.0: Performance Standards - Wastewater. Issue oriented monitoring and follow-up actions for various incidents are outlined in Table 7.15.

2. When a violation of the prescribed effluent standard occurs, the owner should:

   (i) notify AENV in accordance with Section 7.4;

   (ii) determine the cause of the problem; and

   (iii) take action as directed by AENV.

7.2.4.1 Activated Sludge System

TABLE 7.15
ISSUE ORIENTED AND FOLLOW-UP MONITORING

Incident	Parameter	Point of Measurement	Type of Sample	Monitoring Frequency	Follow-Up
Plant by-pass	flow	By-pass line	N/A	Each incident	-
	CBOD		Grab/composite		
	TSS		Grab/composite		
	Coliform (total)		Grab		
	Coliform (fecal)		Grab		
	NH_3		Grab/composite		
	Total.P		Grab/composite		
Sludge bulking	CBOD (primary)	Downstream of primary clarifier	Grab/composite	For the duration of event	Adjust sludge age, adjust D.O levels, Adjust WAS rates, chlorination of RAS
	TSS (primary)	Downstream of primary clarifier	Grab/composite		
	pH (primary))	Downstream of primary clarifier	Grab/composite		
	MLSS	aeration tank	Grab		
	D.O.	aeration tank	N/A		
	NO_3-N	RAS line	Grab		
Rising Sludge/ Denitrification	CBOD (primary)	Downstream of primary clarifier	Grab/composite	For the duration of the event	Increase F/M ratio, Adjust RAS, Adjust D.O. levels, Step-feed influent
	TSS (primary)	Downstream of primary clarifier	Grab/composite		
	MLSS	aeration tank	Grab		
	NO_3-N	RAS line	Grab		
Primary Sludge Septicity	Flow	Primary sludge line	N/A	For the duration of the event	Increase sludge wasting, monitor retention time
	Retention time				
	Density				
	CBOD (raw)				
	TSS (raw)				

7.3 Record Keeping

All records should bear the signature of the operator in responsible charge of the wastewater system or his or her representative. Owners shall keep these records available for inspection by AENV and shall send the records to AENV if requested.

The owners shall keep the following records and effluent quality analyses:

1. All daily records for treatment performance and compliance monitoring for five years.
2. All daily records for operational monitoring for three years.
3. Actual laboratory reports may be kept or data may be transferred to tabular summaries, provided the following information is included:
    i) the date, place and time of sampling, and the name of the person collecting the sample;
    ii) identification of the sample type (compliance sample, operational sample, special purpose sample);
    iii) date of analysis;
    iv) laboratory and person responsible for performing analysis;
    v) the analytical method used; and
    vi) the results of the analysis.
4. Records of action taken by the system to correct violations of effluent standards or operating approval requirements.
5. Copies of project reports, construction documents and related drawings, inspection reports and approvals should be kept for the life of the facility.

7.4 Reporting

1. Reporting requirements shall be in accordance with the operating approval for the facility, issued by AENV.
2. The owner shall report to AENV within one week:
    i) the violation of prescribed effluent standards for that facility; and
    ii) the failure to comply with the treatment performance and compliance monitoring requirements.

3. Immediate notification by telephone to 1-780-422-4505 shall be made, followed by a written report to AENV within one week and remedial action carried out as per EPEA Division 1, Sections 110, 111 and 112 in the event of:

   - any discharge of raw or partially treated wastewater
   - any unauthorized discharge or accidental spill of raw or partially treated wastewater
   - any overflow from the wastewater collection or treatment system.

4. At least one week prior to draining of wastewater stabilization ponds, the owner shall notify AENV in writing of the proposed discharging schedule.

5. The owner should compile and submit an annual report on or before February 28 of the following year on which the information was collected. The report should include the following:

   i) a monthly summary of all operational and compliance monitoring for that particular facility, as identified by AENV;

   ii) a summary of approval contraventions and remedial measures taken; and

   iii) a summary of any permanent upgrading works undertaken during the year.

Appendices

APPENDIX A

CT VALUES FOR INACTIVATION OF GIARDIA CYSTS BY FREE CHLORINE

Adopted from Optimizing Water Treatment
Plant Performance Using the Composite
Correction Program (1991), prepared by
Process Applications Inc., Fort Collins,
Colo., for the US Environmental Protection
Agency, Office of Drinking Water,
Cincinnati, Ohio

TABLE A-1

CT VALUES FOR INACTIVATION OF GIARDIA CYSTS BY FREE CHLORINE AT 0.5°C OR LOWER

Chlorine Concentrations (mg/L)	pH < = 6 Log Inactivations						pH = 6.5 Log Inactivations						pH = 7.0 Log Inactivations						pH = 7.5 Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0
< = 0.4	23	46	69	91	114	137	27	54	82	109	136	163	33	65	98	130	163	195	40	79	119	158	198	237
0.6	24	47	71	94	118	141	28	56	84	112	140	168	33	67	100	133	167	200	40	80	120	159	199	239
0.8	24	48	73	97	121	145	29	57	86	115	143	172	34	68	103	137	171	205	41	82	123	164	205	246
1	25	49	74	99	123	148	29	59	88	117	147	176	35	70	105	140	175	210	42	84	127	169	211	253
1.2	25	51	76	101	127	152	30	60	90	120	150	180	36	72	108	143	179	215	43	86	130	173	216	259
1.4	26	52	78	103	129	155	31	61	92	123	153	184	37	74	111	147	184	221	44	89	133	177	222	266
1.6	26	52	79	105	131	157	32	63	95	126	158	189	38	75	113	151	188	226	46	91	137	182	228	273
1.8	27	54	81	108	135	162	32	64	97	129	161	193	39	77	116	154	193	231	47	93	140	186	233	279
2	28	55	83	110	138	165	33	66	99	131	164	197	39	79	118	157	197	236	48	95	143	191	238	286
2.2	28	56	85	113	141	169	34	67	101	134	168	201	40	81	121	161	202	242	50	99	149	198	248	297
2.4	29	57	86	115	143	172	34	68	103	137	171	205	41	82	124	165	206	247	50	99	149	199	248	298
2.6	29	58	88	117	146	175	35	70	105	139	174	209	42	84	126	168	210	252	51	101	152	203	253	304
2.8	30	59	89	119	148	178	36	71	107	142	178	213	43	86	129	171	214	257	52	103	155	207	258	310
3	30	60	91	121	151	181	36	72	109	145	181	217	44	87	131	174	218	261	53	105	158	211	263	316

Chlorine Concentrations (mg/L)	pH = 8.0 Log Inactivations						pH = 8.5 Log Inactivations						pH < = 9.0 Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0
< = 0.4	46	92	139	185	231	277	55	110	165	219	274	329	65	130	195	260	325	390
0.6	48	95	143	191	238	286	57	114	171	228	285	342	68	136	204	271	339	407
0.8	49	98	148	197	246	295	59	118	177	236	295	354	70	141	211	281	352	422
1	51	101	152	203	253	304	61	122	183	243	304	365	73	146	219	291	364	437
1.2	52	104	157	209	261	313	63	125	188	251	313	376	75	150	226	301	376	451
1.4	54	107	161	214	268	321	65	129	194	258	323	387	77	155	232	309	387	464
1.6	55	110	165	219	274	329	66	132	199	265	331	397	80	159	239	318	398	477
1.8	56	113	169	225	282	338	68	136	204	271	339	407	82	163	245	326	408	489
2	58	115	173	231	288	346	70	139	209	278	348	417	83	167	250	333	417	500
2.2	59	118	177	235	294	353	71	142	213	284	355	426	85	170	256	341	426	511
2.4	60	120	181	241	301	361	73	145	218	290	363	435	87	174	261	348	435	522
2.6	61	123	184	245	307	368	74	148	222	296	370	444	89	178	267	355	444	533
2.8	63	125	188	250	313	375	75	151	226	301	377	452	91	181	272	362	453	543
3	64	127	191	255	318	382	77	153	230	307	383	460	92	184	276	368	460	552

TABLE A-2
CT VALUES FOR INACTIVATION OF GIARDIA CYSTS BY FREE CHLORINE AT 5^{O}C

Chlorine Concentrations (mg/L)	pH < = 6 Log Inactivations						pH = 6.5 Log Inactivations						pH = 7.0 Log Inactivations						pH = 7.5 Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0
< = 0.4	16	32	49	65	81	97	20	39	59	78	98	117	23	46	70	93	116	139	28	55	83	111	138	166
0.6	17	33	50	67	83	100	20	40	60	80	100	120	24	48	72	95	119	143	29	57	86	114	143	171
0.8	17	34	52	69	86	103	20	41	61	81	102	122	24	49	73	97	122	146	29	58	88	117	146	175
1	18	35	53	70	88	105	21	42	63	83	104	125	25	50	75	99	124	149	30	60	90	119	149	179
1.2	18	36	54	71	89	107	21	42	64	85	106	127	25	51	76	101	127	152	31	61	92	122	153	183
1.4	18	36	55	73	91	109	22	43	65	87	108	130	26	52	78	103	129	155	31	62	94	125	156	187
1.6	19	37	56	74	93	111	22	44	66	88	110	132	26	53	79	105	132	158	32	64	96	128	160	192
1.8	19	38	57	76	95	114	23	45	68	90	113	135	27	54	81	108	135	162	33	65	98	131	163	196
2	19	39	58	77	97	116	23	46	69	92	115	138	28	55	83	110	138	165	33	67	100	133	167	200
2.2	20	39	59	79	98	118	23	47	70	93	117	140	28	56	85	113	141	169	34	68	102	136	170	204
2.4	20	40	60	80	100	120	24	48	72	95	119	143	29	57	86	115	143	172	35	70	105	139	174	209
2.6	20	41	61	81	102	122	24	49	73	97	122	146	29	58	88	117	146	175	36	71	107	142	178	213
2.8	21	41	62	83	103	124	25	49	74	99	123	148	30	59	89	119	148	178	36	72	109	145	181	217
3	21	42	63	84	105	126	25	50	76	101	126	151	30	61	91	121	152	182	37	74	111	147	184	221

Chlorine Concentrations (mg/L)	pH < =8.0 Log Inactivations						pH = 8.5 Log Inactivations						pH < = 9.0 Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0
< = 0.4	33	66	99	132	165	198	39	79	118	157	197	236	47	93	140	186	233	279
0.6	34	68	102	136	170	204	41	81	122	163	203	244	49	97	146	194	243	291
0.8	35	70	105	140	175	210	42	84	126	168	210	252	50	100	151	201	251	301
1	36	72	108	144	180	216	43	87	130	173	217	260	52	104	156	208	260	312
1.2	37	74	111	147	184	221	45	89	134	178	223	267	53	107	160	213	267	320
1.4	38	76	114	151	189	227	46	91	137	183	228	274	55	110	165	219	274	329
1.6	39	77	116	155	193	232	47	94	141	187	234	281	56	112	169	225	281	337
1.8	40	79	119	159	198	238	48	96	144	191	239	287	58	115	173	230	288	345
2	41	81	122	162	203	243	49	98	147	196	245	294	59	118	177	235	294	353
2.2	41	83	124	16	5207	248	50	100	150	200	250	300	60	120	181	241	301	361
2.4	42	84	127	169	211	253	51	102	153	204	255	306	61	123	184	245	307	368
2.6	43	86	129	172	215	258	52	104	156	208	260	312	63	125	188	250	313	375
2.8	44	88	432	175	219	263	53	106	159	212	265	318	64	127	191	255	318	382
3	45	89	134	179	223	268	54	108	162	216	270	324	65	130	195	259	324	389

TABLE A-3

CT VALUES FOR INACTIVATION OF GIARDIA CYSTS BY FREE CHLORINE AT 10°C

Chlorine Concentrations (mg/L)	pH < = 6 Log Inactivations						pH = 6.5 Log Inactivations						pH = 7.0 Log Inactivations						pH = 7.5 Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0
< = 0.4	12	24	37	49	61	73	15	29	44	59	73	88	17	35	52	69	87	104	21	42	63	83	104	125
0.6	13	25	38	50	63	75	15	30	45	60	75	90	18	36	54	71	89	107	21	43	64	85	107	128
0.8	13	26	39	52	65	78	15	31	46	61	77	92	18	37	55	73	92	110	22	44	66	87	109	131
1	13	26	40	53	66	79	16	31	47	63	78	94	19	37	56	75	93	112	22	45	67	89	112	134
1.2	13	27	40	53	67	80	16	32	48	63	79	95	19	38	57	76	95	114	23	46	69	91	114	137
1.4	14	27	41	55	68	82	16	33	49	65	82	98	19	39	58	77	97	116	23	47	70	93	117	140
1.6	14	28	42	55	69	83	17	33	50	66	83	99	20	40	60	79	99	119	24	48	72	96	120	144
1.8	14	29	43	57	72	86	17	34	51	67	84	101	20	41	61	81	102	122	25	49	74	98	123	147
2	15	29	44	58	73	87	17	35	52	69	87	104	21	41	62	83	103	124	25	50	75	100	125	150
2.2	15	30	45	59	74	89	18	35	53	70	88	105	21	42	64	85	106	127	26	51	77	102	128	153
2.4	15	30	45	60	75	90	18	36	54	71	89	107	22	43	65	86	108	129	26	52	79	105	131	157
2.6	15	31	46	61	77	92	18	37	55	73	92	110	22	44	66	87	109	131	27	53	80	107	133	160
2.8	16	31	47	62	78	93	19	37	56	74	93	111	22	45	67	89	112	134	27	54	82	109	136	163
3	16	32	48	63	79	95	19	38	57	75	94	113	23	46	69	91	114	137	28	55	83	111	138	166

Chlorine Concentrations (mg/L)	pH < =8.0 Log Inactivations						pH = 8.5 Log Inactivations						pH < = 9.0 Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0
< = 0.4	25	50	75	99	124	149	30	59	89	118	148	177	35	70	105	139	174	209
0.6	26	51	77	102	128	153	31	61	92	122	153	183	36	73	109	145	182	218
0.8	26	53	79	105	132	158	32	63	95	126	158	189	38	75	113	151	188	226
1	27	54	81	108	135	162	33	65	98	130	163	195	39	78	117	156	195	234
1.2	28	55	83	111	138	166	33	67	100	133	167	200	40	80	120	160	200	240
1.4	28	57	85	113	142	170	34	69	103	137	172	206	41	82	124	165	206	247
1.6	29	58	87	116	145	174	35	70	106	141	176	211	42	84	127	169	211	253
1.8	30	60	90	119	149	179	36	72	108	143	179	215	43	86	130	173	216	259
2	30	61	91	121	152	182	37	74	111	147	184	221	44	88	133	177	221	265
2.2	31	62	93	124	155	186	38	75	113	150	188	225	45	90	136	181	226	271
2.4	32	63	95	127	158	190	38	77	115	153	192	230	46	92	138	184	230	276
2.6	32	65	97	129	162	194	39	78	117	156	195	234	47	94	141	187	234	281
2.8	33	66	99	131	164	197	40	80	120	159	199	239	48	96	144	191	239	287
3	34	67	101	134	168	201	41	81	122	162	203	243	49	97	146	195	243	292

TABLE A-4
CT VALUES FOR INACTIVATION OF GIARDIA CYSTS BY FREE CHLORINE AT 15°C

Chlorine Concentrations (mg/L)	pH < = 6 Log Inactivations						pH = 6.5 Log Inactivations						pH = 7.0 Log Inactivations						pH = 7.5 Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0
< = 0.4	8	16	25	33	41	49	10	20	30	39	49	59	12	23	35	47	58	70	14	28	42	55	69	83
0.6	8	17	25	33	42	50	10	20	30	40	50	60	12	24	36	48	60	72	14	29	43	57	72	86
0.8	9	17	26	35	43	52	10	20	31	41	51	61	12	24	37	49	61	73	15	29	44	59	73	88
1	9	18	27	35	44	53	11	21	32	42	53	63	13	25	38	50	63	75	15	30	45	60	75	90
1.2	9	18	27	36	45	54	11	21	32	43	53	64	13	25	38	51	63	76	15	31	46	61	77	92
1.4	9	18	28	37	46	55	11	22	33	43	54	65	13	26	39	52	65	78	16	31	47	63	78	94
1.6	9	19	28	37	47	56	11	22	33	44	55	66	13	26	40	53	66	79	16	32	48	64	80	96
1.8	10	19	29	38	48	57	11	23	34	45	57	68	14	27	41	54	68	81	16	33	49	65	82	98
2	10	19	29	39	48	58	12	23	35	46	58	69	14	28	42	55	69	83	17	33	50	67	83	100
2.2	10	20	30	39	49	59	12	23	35	47	58	70	14	28	43	57	71	85	17	34	51	68	85	102
2.4	10	20	30	40	50	60	12	24	36	48	60	72	14	29	43	57	72	86	18	35	53	70	88	105
2.6	10	20	31	41	51	61	12	24	37	49	61	73	15	29	44	59	73	88	18	36	54	71	89	107
2.8	10	21	31	41	52	62	1	225	67	49	62	74	15	30	45	59	74	89	18	36	55	73	91	109
3	11	2	132	42	53	63	13	25	38	51	63	76	15	30	46	61	76	91	19	37	56	74	93	111

Chlorine Concentrations (mg/L)	pH < =8.0 Log Inactivations						pH = 8.5 Log Inactivations						pH < = 9.0 Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0
< = 0.4	17	33	50	66	83	99	20	39	59	79	98	118	23	47	70	93	117	140
0.6	17	34	51	68	85	102	20	41	61	81	102	122	24	49	73	97	122	146
0.8	18	35	53	70	88	105	21	42	63	84	105	126	25	50	76	101	126	151
1	18	36	54	72	90	108	22	43	65	87	108	130	26	52	78	104	130	156
1.2	19	37	56	74	93	111	22	45	67	89	112	134	27	53	80	107	133	160
1.4	19	38	57	76	95	114	23	46	69	91	114	137	28	55	83	110	138	165
1.6	19	39	58	77	97	116	24	47	71	94	118	141	28	56	85	113	141	169
1.8	20	40	60	79	99	119	24	48	72	96	120	144	29	58	87	115	144	173
2	20	41	61	81	102	122	25	49	74	98	123	147	30	59	89	118	148	177
2.2	21	41	62	83	103	124	25	50	75	100	125	150	30	60	91	121	151	181
2.4	21	42	64	85	106	127	26	51	77	102	128	153	31	61	92	123	153	184
2.6	22	43	65	86	108	129	26	52	78	104	130	156	31	63	94	125	157	188
2.8	22	44	66	88	110	132	27	53	80	106	133	159	32	64	96	127	159	191
3	22	45	67	89	112	134	27	54	81	108	135	162	33	65	98	130	163	195

TABLE A-5
CT VALUES FOR INACTIVATION OF GIARDIA CYSTS BY FREE CHLORINE AT 20°C

Chlorine Concentrations (mg/L)	pH < = 6 Log Inactivations						pH = 6.5 Log Inactivations						pH = 7.0 Log Inactivations						pH = 7.5 Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0
< = 0.4	6	12	18	24	30	36	7	15	22	29	37	44	9	17	26	35	43	52	10	21	31	41	52	62
0.6	6	13	19	25	32	38	8	15	23	30	38	45	9	18	27	36	45	54	11	21	32	43	53	64
0.8	7	13	20	26	33	39	8	15	23	31	38	46	9	18	28	37	46	55	11	22	33	44	55	66
1	7	13	20	26	33	39	8	16	24	31	39	47	9	19	28	37	47	56	11	22	34	45	56	67
1.2	7	13	20	27	33	40	8	16	24	32	40	48	10	19	29	38	48	57	12	23	35	46	58	69
1.4	7	14	21	27	34	41	8	16	25	33	41	49	10	19	29	39	48	58	12	23	35	47	58	70
1.6	7	14	21	28	35	42	8	17	25	33	42	50	10	20	30	39	49	59	12	24	36	48	60	72
1.8	7	14	22	29	36	43	9	17	26	34	43	51	10	20	31	41	51	61	12	25	37	49	62	74
2	7	15	22	29	37	44	9	17	26	35	43	52	10	21	31	41	52	62	13	25	38	50	63	75
2.2	7	15	22	29	37	44	9	18	27	35	44	53	11	21	32	42	53	63	13	26	39	51	64	77
2.4	8	15	23	30	38	45	9	18	27	36	45	54	11	22	33	43	54	65	13	26	39	52	65	78
2.6	8	15	23	31	38	46	9	18	28	37	46	55	11	22	33	44	55	66	13	27	40	53	67	80
2.8	8	16	24	31	39	47	9	19	28	37	47	56	11	22	34	45	56	67	14	27	41	54	68	81
3	8	16	24	31	39	47	10	19	29	38	48	57	11	23	34	45	57	68	14	28	42	55	69	83

Chlorine Concentrations (mg/L)	pH < =8.0 Log Inactivations						pH = 8.5 Log Inactivations						pH < = 9.0 Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0
< = 0.4	12	25	37	49	62	74	15	30	45	59	74	89	18	35	53	70	88	105
0.6	13	26	39	51	64	77	15	31	46	61	77	92	18	36	55	73	91	109
0.8	13	26	40	53	66	79	16	32	48	63	79	95	19	38	57	75	94	113
1	14	27	41	54	68	81	16	33	49	65	82	98	20	39	59	78	98	117
1.2	14	28	42	55	69	83	17	33	50	67	83	100	20	40	60	80	100	120
1.4	14	28	43	57	71	85	17	34	52	69	86	103	21	41	62	82	103	123
1.6	15	29	44	58	73	87	18	35	53	70	88	105	21	42	63	84	105	126
1.8	15	30	45	59	74	89	18	36	54	72	90	108	22	43	65	86	108	129
2	15	30	46	61	76	91	18	37	55	73	92	110	22	44	66	88	110	132
2.2	16	31	47	62	78	93	19	38	57	75	94	113	23	45	68	90	113	135
2.4	16	32	48	63	79	95	19	38	58	77	96	115	23	46	69	92	115	138
2.6	16	32	49	65	81	97	20	39	59	78	98	117	24	47	71	94	118	141
2.8	17	33	50	66	83	99	20	40	60	79	99	119	24	48	72	95	119	143
3	17	34	51	67	84	101	20	41	61	81	102	122	24	49	73	97	122	146

TABLE A-6
CT VALUES FOR INACTIVATION OF GIARDIA CYSTS BY FREE CHLORINE AT 25^{O}C

Chlorine Concentrations (mg/L)	pH < = 6						pH = 6.5						pH = 7.0						pH = 7.5					
	Log Inactivations						Log Inactivations						Log Inactivations						Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0
< = 0.4	4	8	12	16	20	24	5	10	15	19	24	29	6	12	18	23	29	35	7	14	21	28	35	42
0.6	4	8	13	17	21	25	5	10	15	20	25	30	6	12	18	24	30	36	7	14	22	29	36	43
0.8	4	9	13	17	22	26	5	10	16	21	26	31	6	12	19	25	31	37	7	15	22	29	37	44
1	4	9	13	17	22	26	5	10	16	21	26	31	6	12	19	25	31	37	8	15	23	30	38	45
1.2	5	9	14	18	23	27	5	11	16	21	27	32	6	13	19	25	32	38	8	15	23	31	38	46
1.4	5	9	14	18	23	27	6	11	17	22	28	33	7	13	20	26	33	39	8	16	24	31	39	47
1.6	5	9	14	19	23	28	6	11	17	22	28	33	7	13	20	27	33	40	8	16	24	32	40	48
1.8	5	10	15	19	24	29	6	11	17	23	28	34	7	14	21	27	34	41	8	16	25	33	41	49
2	5	10	15	19	24	29	6	12	18	23	29	35	7	14	21	27	34	41	8	17	25	33	42	50
2.2	5	10	15	20	25	30	6	12	18	23	29	35	7	14	21	28	35	42	9	17	26	34	43	51
2.4	5	10	15	20	25	30	6	12	18	24	30	36	7	14	22	29	36	43	9	17	26	35	43	52
2.6	5	10	16	21	26	31	6	12	19	25	31	37	7	15	22	29	37	44	9	18	27	35	44	53
2.8	5	10	16	21	26	31	6	12	19	25	31	37	8	15	23	30	38	45	9	18	27	36	45	54
3	5	11	16	21	27	32	6	13	19	25	32	38	8	15	23	31	38	46	9	18	28	37	46	55

Chlorine Concentrations (mg/L)	pH < =8.0						pH = 8.5						pH < = 9.0					
	Log Inactivations						Log Inactivations						Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0
< = 0.4	8	17	25	33	42	50	10	20	30	39	49	59	12	23	35	47	58	70
0.6	9	17	26	34	43	51	10	20	31	41	51	61	12	24	37	49	61	73
0.8	9	18	27	35	44	53	11	21	32	42	53	63	13	25	38	50	63	75
1	9	18	27	36	45	54	11	22	33	43	54	65	13	26	39	52	65	78
1.2	9	18	28	37	46	55	11	22	34	45	56	67	13	27	40	53	67	80
1.4	10	19	29	38	48	57	12	23	35	46	58	69	14	27	41	55	68	82
1.6	10	19	29	39	48	58	12	23	35	47	58	70	14	28	42	56	70	84
1.8	10	20	30	40	50	60	12	24	36	48	60	72	14	29	43	57	72	86
2	10	20	31	41	51	61	12	25	37	49	62	74	15	29	44	59	73	88
2.2	10	21	31	41	52	62	13	25	38	50	63	75	15	30	45	60	75	90
2.4	11	21	32	42	53	63	13	26	39	51	64	77	15	31	46	61	77	92
2.6	11	22	33	43	54	65	13	26	39	52	65	78	16	31	47	63	78	94
2.8	11	22	33	44	55	66	13	27	40	53	67	80	16	32	48	64	80	96
3	11	22	34	45	56	67	14	27	41	54	68	81	16	32	49	65	81	97

APPENDIX B

CT VALUES FOR INACTIVATION OF GIARDIA CYSTS AND VIRUSES BY VARIOUS DISINFECTANTS

Adopted from Straw man Regulations for
Ground Water Disinfection, US Environmental
Protection Agency, Office of Drinking Water,
Washington, D.C. (June 1990)

TABLE B-1
CT VALUES FOR INACTIVATION OF VIRUSES BY FREE CHLORINE*+

	Log Inactivation					
	2.0		3.0		4.0	
	pH		pH		pH	
Temperature (°C)	6-9	10	6-9	10	6-9	10
0.5	6	45	9	66	12	90
5	4	30	6	44	8	60
10	3	22	4	33	6	45
15	2	15	3	22	4	30
20	1	11	2	16	3	22
25	1	7	1	11	2	15

*Data adapted from Sobsey (1988) for inactivation of Hepatitis A virus (HAV) at pH = 6, 7, 8, 9 and 10 and temperature = 5°C. CT values include a safety factor of 3.

+CT values adjusted to other temperatures by doubling CT for each 10°C drop in temperature.

TABLE B-2
CT VALUES FOR INACTIVATION OF GIARDIA CYSTS BY CHLORINE DIOXIDE +*

	Temperature (°C)					
Inactivation	≤1	5	10	15	20	25
0.5 log	10	4.3	4	3.2	2.5	2
1 log	21	8.7	7.7	6.3	5	3.7
1.5 log	32	13	12	10	7.5	5.5
2 log	42	17	15	13	10	7.3
2.5 log	52	22	19	16	13	9
3 log	63	26	23	19	15	11

*Data adapted from Sobsey (1988) for inactivation of Hepatitis A virus (HAV) at pH = 6.0 and temperature = 5°C. CT values include a safety factor of 2.

+CT values adjusted to other temperatures by doubling CT for each 10°C drop in temperature.

TABLE B-3
CT VALUES FOR INACTIVATION OF VIRUSES BY CHLORINE DIOXIDE PH 6-9*+

	Temperature (°C)					
Inactivation	≤1	5	10	15	20	25
2 log	8.4	5.6	4.2	2.8	2.1	1.4
3 log	25.6	17.1	12.8	8.6	6.4	4.3
4 log	50.1	33.4	25.1	16.7	12.5	8.4

*Data adapted from Sobsey (1988) for inactivation of Hepatitis A virus (HAV) at pH = 6.0 and temperature = 5°C. CT values include a safety factor of 2.

+CT values adjusted to other temperatures by doubling CT for each 10°C drop in temperature.

TABLE B-4
CT VALUES FOR INACTIVATION OF GIARDIA CYSTS BY OZONE *+

	Temperature (°C)					
Inactivation	≤1	5	10	15	20	25
0.5 log	0.48	0.32	0.23	0.16	0.1	0.08
1 log	0.97	0.63	0.48	0.32	0.2	0.16
1.5 log	1.5	0.95	0.72	0.48	0.36	0.24
2 log	1.9	1.3	0.95	0.63	0.48	0.32
2.5	2.4	1.6	1.2	0.79	0.6	0.4
3 log	2.9	1.9	1.43	0.95	0.72	0.48

*Data adapted from Sobsey (1988) for inactivation of Hepatitis A virus (HAV) at pH = 6.0 and temperature = 5°C. CT values include a safety factor of 2.

+CT values adjusted to other temperatures by doubling CT for each 10°C drop in temperature.

TABLE B-5
CT VALUES FOR INACTIVATION OF VIRUSES BY OZONE*+

	Temperature (°C)					
Inactivation	≤1	5	10	15	20	25
2 log	0.9	0.6	0.5	0.3	0.25	0.15
3 log	1.4	0.9	0.8	0.5	0.4	0.25
4 log	1.8	1.2	1.0	0.6	0.5	0.3

*Data adapted from Roy (1982) for inactivation of poliovirus for pH = 7.2 and temperature = 5°C. CT values include a safety factor of 3.

+CT values adjusted to other temperatures by doubling CT for each 10°C drop in temperature.

TABLE B-6
CT VALUES FOR INACTIVATION OF GIARDIA CYSTS BY CHLORAMINE pH 6-9*+

	Temperature (°C)					
Inactivation	**≤1**	**5**	**10**	**15**	**20**	**25**
0.5 log	635	365	310	250	185	125
1 log	1270	735	615	500	370	250
1.5 log	1900	1100	930	750	550	375
2 log	2535	1470	1230	1000	735	500
2.5	3170	1830	1540	1250	915	625
3 log	3800	2200	1850	1500	1100	750

*Data adapted from Sobsey (1988) for inactivation of Hepatitis A virus (HAV) at pH = 6.0 and temperature = 5°C. CT values include a safety factor of 2.

+CT values adjusted to other temperatures by doubling CT for each 10°C drop in temperature.

TABLE B-7
CT VALUES FOR INACTIVATION OF VIRUSES BY CHLORAMINE*+

	Temperature (°C)					
Inactivation	**≤1**	**5**	**10**	**15**	**20**	**25**
2 log	1243	857	643	428	321	214
3 log	2063	1423	1067	712	534	356
4 log	2883	1988	1491	994	746	497

*Data adapted from Roy (1982) for inactivation of poliovirus for pH = 7.2 and temperature = 5°C. CT values include a safety factor of 3.

+CT values adjusted to other temperatures by doubling CT for each 10°C drop in temperature.

APPENDIX C

IT VALUES FOR INACTIVATION OF GIARDIA CYSTS, CRYPTOSPORIDIUM OOCYSTS AND VIRUSES BY UV LIGHT

Adopted from Ultraviolet Disinfection Guidance Manual, US Environmental
Protection Agency, Office of Drinking Water,
Washington, D.C. (June 2003, Draft)

UV Dose Requirements Used During Validation Testing[1]

	Log Inactivation							
	0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0
Cryptosporidium	1.6	2.5	3.9	5.8	8.5	12	-	-
Giardia	1.5	2.1	3.0	5.2	7.7	11	-	-
Virus	39	58	79	100	121	143	163	186

[1] 40 CFR 141.729(d)

Note: The above IT doses are based on collimated beam bench tests. In determining the actual dosage for individual reactors, various factors need to be applied as per the USEPA, DVGW W-294, or AWWARF / NWRI standards.

APPENDIX D

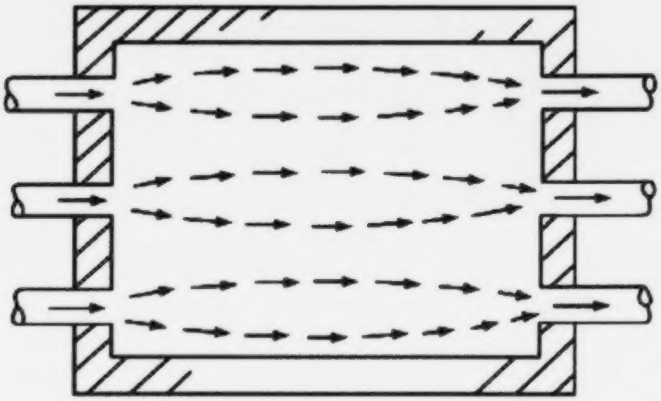
TYPICAL BAFFLING CONDITIONS

TYPICAL BAFFLING CONDITIONS**

Baffling Condition	T_{10}/T Ratio	Baffling Description
Unbaffled (mixed flow)	0.1	None, agitated basin, very low length to width ratio, high inlet and outlet flow velocities
Poor	0.3	Single or multiple unbaffled inlets and outlets, no intra-basin baffles
Average	0.5	Baffled inlet or outlet with some intra-basin baffles
Superior	0.7	Perforated inlet baffle, serpentine or perforated intra-basin baffles, outlet weir or perforated launders
Perfect (plug flow	1.0	Very high length to width ratio (pipeline flow), perforated inlet, outlet, and intra-basin baffles

**Based on "Guidance Manual for Compliance with the Filtration and Disinfection Requirements for Public Water Systems using Surface Water Sources", USEPA, October 1990.

Poor Baffling Conditions - Rectangular Contact Basin



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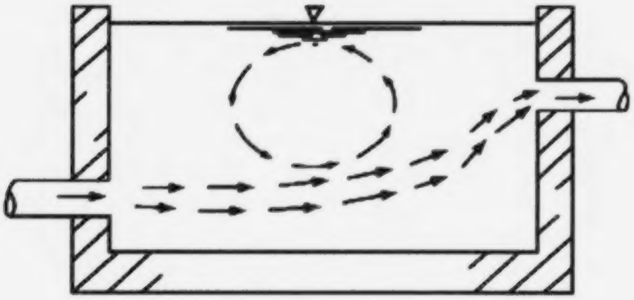
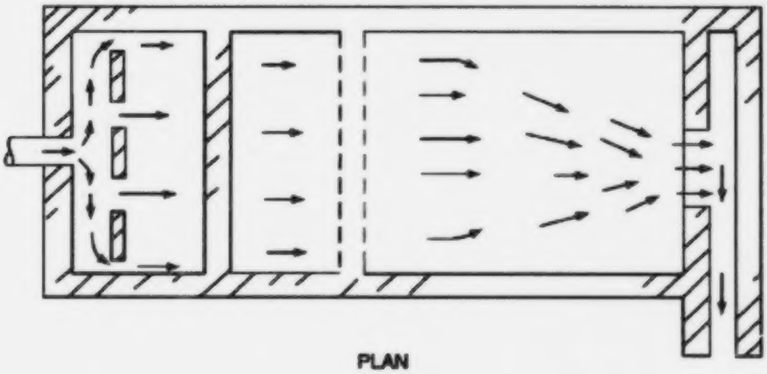


SECTION

Average Baffling Conditions - Rectangular Contact Basin



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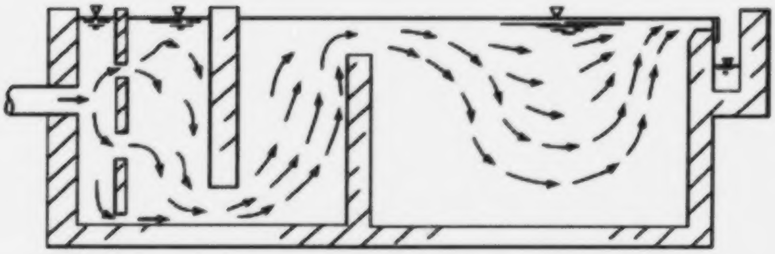

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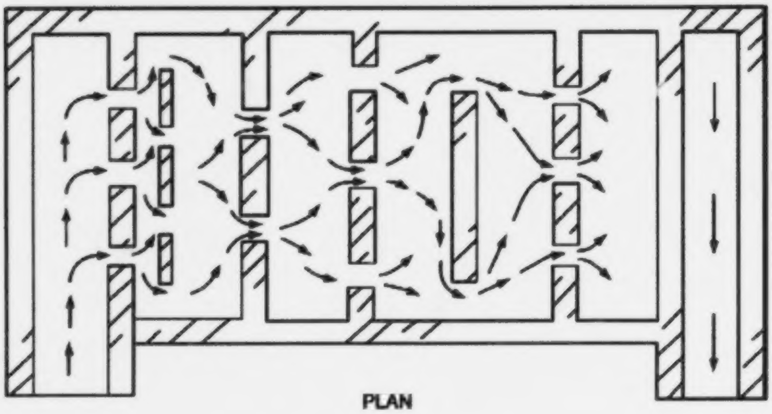
Superior Baffling Conditions - Rectangular Contact Basin



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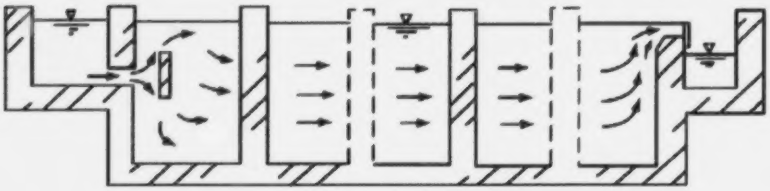

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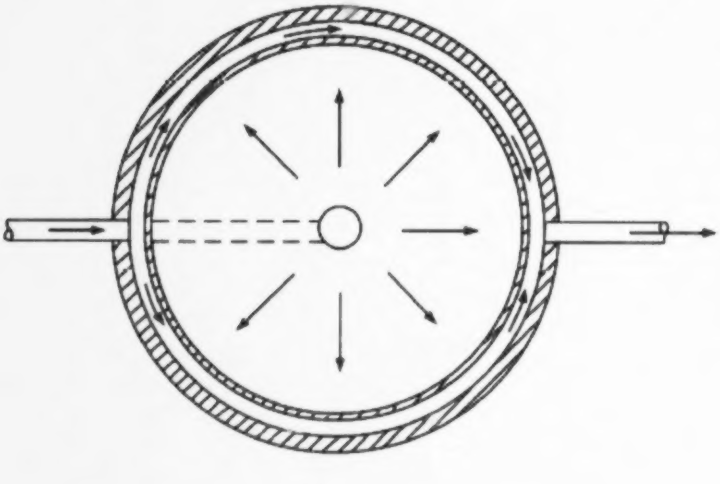
Poor Baffling Conditions - Circular Contact Basin



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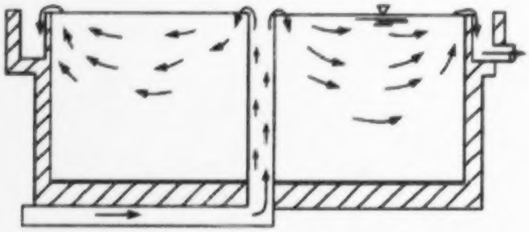

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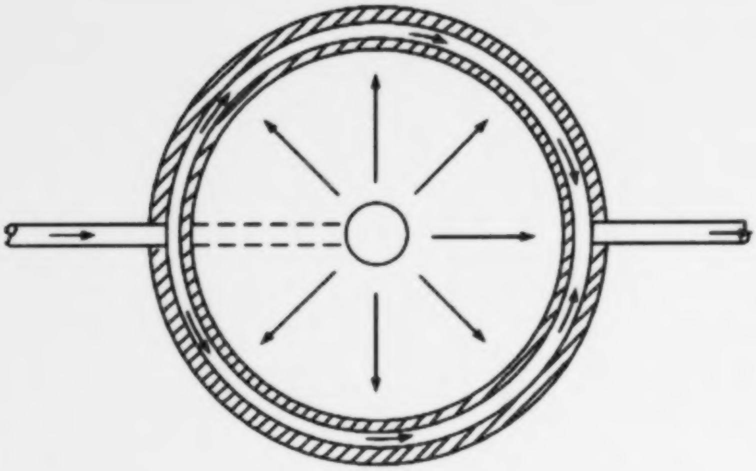
Average Baffling Conditions - Circular Contact Basin



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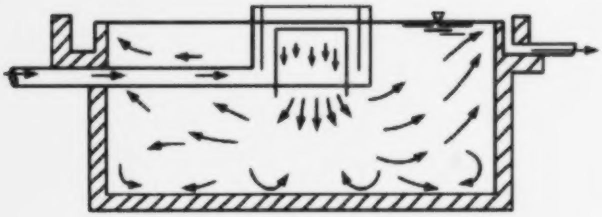

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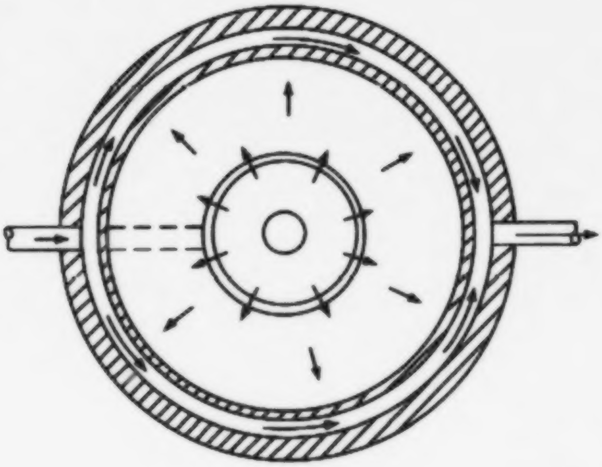
Superior Baffling Conditions - Circular Contact Basin



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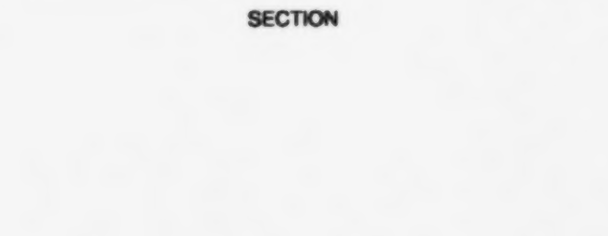


SECTION

APPENDIX E

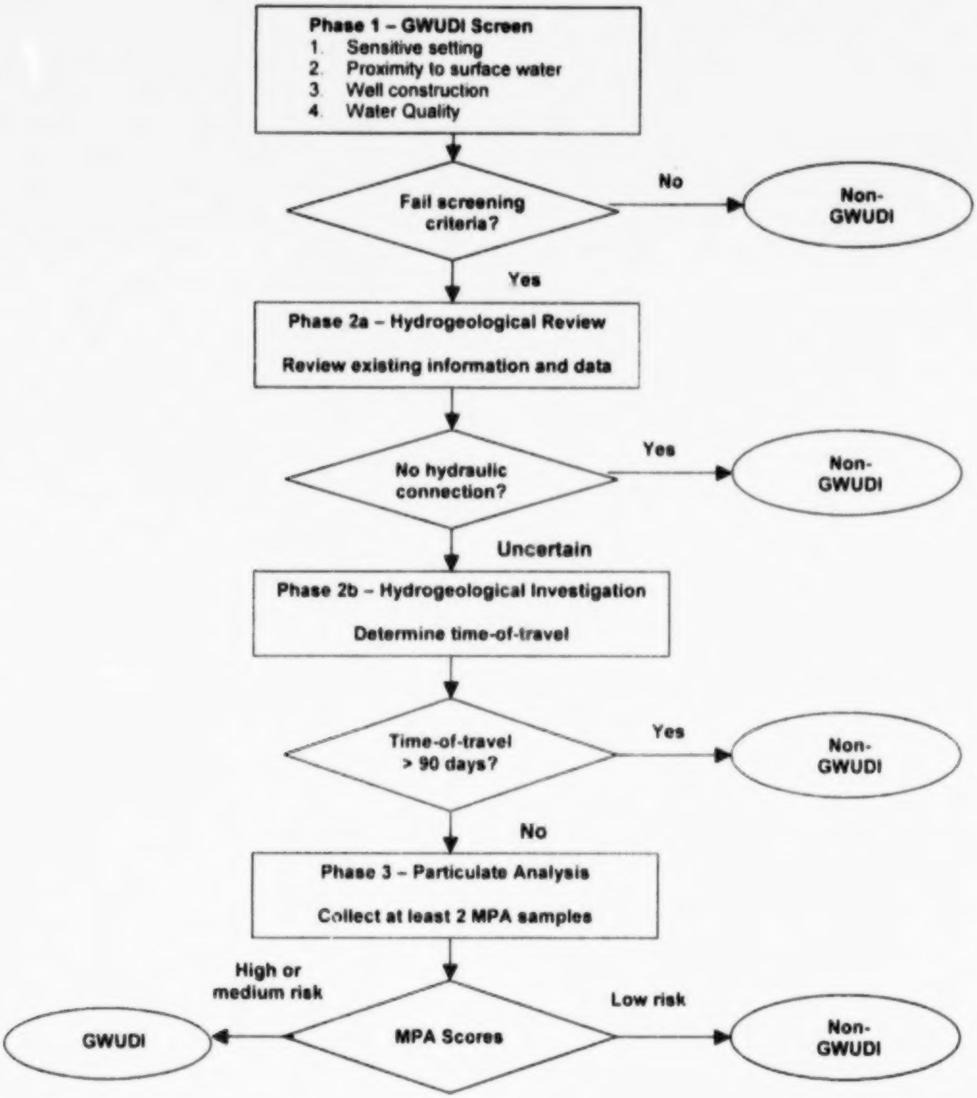
Assessment Guideline for Groundwater Under the Direct Influence of Surface Water (GWUDI)

General

Groundwater under the direct influence of surface water (GWUDI) refers to groundwater supply sources that are vulnerable to contamination by pathogens from nearby surface water or infiltrating precipitation. Groundwater sources that are determined to be GWUDI require treatment equivalent to that required for surface water sources as specified in Section 1.2.1. Waterworks systems using "high quality groundwater" must not be under the direct influence of surface water according to the *Potable Water Regulation (Alberta Regulation 277/2003)*.

This assessment guideline presents the protocol for determining whether a source is GWUDI or non-GWUDI. The assessment is to be carried out by a qualified hydrogeologist or groundwater engineer who is a member of APEGGA. The assessment is divided into three phases, which is outlined in the flow chart in Figure 1. The concept of the guideline is to initially flag potential GWUDI sources, followed by more detailed investigations to determine whether or not a source is GWUDI.

Figure 1: GWUDI Assessment Flowchart


Phase 1 – GWUDI Screen
1. Sensitive setting
2. Proximity to surface water
3. Well construction
4. Water Quality
Fail screening criteria?
No
Non-GWUDI
Yes
Phase 2a – Hydrogeological Review
Review existing information and data
No hydraulic connection?
Yes
Non-GWUDI
Uncertain
Phase 2b – Hydrogeological Investigation
Determine time-of-travel
Time-of-travel > 90 days?
Yes
Non-GWUDI
No
Phase 3 – Particulate Analysis
Collect at least 2 MPA samples
High or medium risk
GWUDI
MPA Scores
Low risk
Non-GWUDI

PHASE 1 – GWUDI Screen

The purpose of the screening is to rapidly identify obvious non-GWUDI sources that do not require a detailed assessment. The screening criteria are as follows:

1. ***Sensitive Setting*** – the source shall not be a:
    a. spring, infiltration gallery, shallow collector system, artificial recharge system, bored well or dug well
    b. well with a production zone less than 15m below ground surface
    c. well in an unconfined aquifer
    d. well completed in fractured or karst bedrock exposed at or near the land surface.
2. ***Proximity to surface water*** – the source shall not be located within 100m of any permanent, intermittent or seasonal surface water body, including ponds, sloughs, lakes, rivers, streams, dugouts, lagoons, reservoirs, irrigation canals or ditches, gravel pits, mining pits or any other open water features.
3. ***Well Construction*** – The source shall be a drilled well which meets the requirements under the current version of the *Alberta Water (Ministerial) Regulation (Alberta Regulation 205/98)*. A drilled well shall have a surface seal that prevents surface water from migrating down the annulus of the well. A drilled well shall be constructed in a manner such that only water from the producing interval enters the well. The wellhead shall be well graded and drained and show no signs of poor construction or deterioration.
4. ***Water Quality*** – The raw or treated water from the source shall not exhibit evidence of contamination by surface water. This means significant occurrence of insects, insect parts, and other microorganisms such as total coliforms (on a regular basis), E.coli, algae, *Giardia*, *Cryptosporidium*, or viruses; or significant and relative rapid shifts in water characteristics such as turbidity, temperature, conductivity, or pH, closely correlating to climatological or surface water conditions.

Should a drinking water source not meet any of the above criteria, the source is flagged as potentially GWUDI, and the assessment is to proceed to Phase 2 or the source can be declared as GWUDI.

If the criteria under #1 and #2 are met but not the criteria under #3 and #4, instead of proceeding to Phase 2, system owners may modify the well construction to ensure the criteria under #3 and/or #4 are met.

PHASE 2a –Hydrogeological Review

The objective the hydrogeological review is to determine if a water source can be designated as GWUDI or non-GWUDI based on existing data and knowledge. It may identify factors not considered in the screening process. The following information is normally required for this evaluation:

- geological / lithological data, including depth and thickness of production zones, overlying confining beds and other subsurface units
- depth of surface water bodies, penetration of any confining units by surface water bodies
- cross-section(s) showing site stratigraphy in relation to surface water bodies
- any history of flooding
- pumping test results (recharge boundaries , hydraulic connection to surface water)
- hydraulic conductivity testing of confining units
- comparison of any historic groundwater level and surface water level monitoring
- comparison of any historic groundwater quality and surface water quality data
- estimate of time-of-travel between surface water and the source where possible.

If there is reasonable uncertainty as to whether a source is vulnerable to the direct influence of surface water, further assessment is required. Proceed to Phase 2b or declare the source as GWUDI.

PHASE 2b –Hydrogeological Investigation

The objective of the hydrogeological investigation is to determine if there is an existing or potential hydraulic connection that could allow rapid recharge of the well by surface water or precipitation. This phase can be combined with Phase 2a under one hydrogeological investigation.

The hydrogeological investigation will require determination of the time-of- travel between a surface water body and a source well. Various methods are available to determine time-of-travel, including water quality hydrograph analysis, computer modeling, analytical methods or tracer tests. The choice of method should take into account the proximity of the surface water body and/or anticipated travel times. For instance, hydrograph analysis of water quality parameters such as temperature, conductivity and pH may be the best option for surface water bodies that are very close to a source well in the same aquifer. It is recommended that monitoring of these parameters be collected on a weekly basis for a minimum of one year, unless a hydraulic connection is recognized early in the program.

Computer modeling involves using particle-tracking techniques to determine time-of-travel, in a similar manner to capture zone modeling. This option may be best suited to situation where greater travel times are anticipated and sufficient information is available. Note that this option may require further intrusive work (e.g. drilling, pumping test, monitoring, etc) in order to obtain suitable information for the modeling work. Model assumptions and sensitivity analysis must be included in the final report.

Should the time-of-travel be determined to be less than 90 days, proceed to Phase 3 or declare the source as GWUDI.

PHASE 3 – Microscopic Particulate Analysis

The results of Phase 2b may determine that there is a hydraulic connection between a source well and a nearby surface water body. However if the subsurface units provide sufficient natural filtration to remove most surface water organisms and debris, the well source may be exempted as GWUDI.

Microscopic Particulate Analysis (MPA) is used to determine if there are significant surface water particulates reaching the well source. The test involves filtering approximately 4,500 litres of water to concentrate organisms and debris, which are then identified and quantified under a microscope by an accredited laboratory. The laboratory shall classify the result as low, medium or high risk.

A minimum of two MPA samples shall be collected, during periods when there is the greatest possibility for surface water to impact a source well (i.e. worst-case situation). This will usually be after a significant storm or snow melt event. *The sampling time can only be determined after the time-of-travel has been determined under Phase 2.* If lag time is not used to determine the sampling times, there is a strong possibility that the result will not reflect a worst-case situation. It is recommended that at least one sample be collected in the spring.

The MPA analysis and scoring is to be conducted according to the *Consensus Method for Determining Groundwaters Under the Direct Influence of Surface Water Using Microscopic Particulate Analysis* (USEPA, 1992). Under this method, samples are scored as follows:

<10	low risk
10 – 19	medium risk
>20	high risk.

A water well source shall be declared GWUDI upon a ***medium*** or ***high-risk*** score, unless remedial action and/or further sampling demonstrate otherwise.

GWUDI Determination

A qualified hydrogeologist or groundwater engineer who is a member of APEGGA shall conduct GWUDI assessments. Professional judgment shall be used to evaluate all the evidence collected in the final determination of whether a water source is GWUDI or non-GWUDI. Generally, all well sources that that do not exhibit any evidence of a current or potential direct connection with surface water or are determined to have a time-of-travel greater than or equal to 90 days to any nearby surface water bodies will be considered non-GWUDI. Evidence for a well source being GWUDI is generally more conclusive than evidence it is not GWUDI. Where uncertainty or doubt exists, it is best to adopt a cautionary approach and consider the source GWUDI.

References

AWWARF (American Water Works Association Research Foundation). 2001. *Investigation of Criteria for GWUDI Determination.*

Nova Scotia Department of Environment and Labour. 2002. *Protocol for Determining Groundwater Under the Direct Influence of Surface Water.*

Ontario Ministry of Environment. 2001. *Terms of Reference: Hydrogeological Study to Examine Groundwater Sources Potentially Under the Direct Influence of Surface Water.*

Ontario Ministry of Environment. 2001. *Delineation of Wellhead Protection Areas for Municipal Groundwater Supply Wells Under the Direct Influence of Surface Water.*

Saskatchewan Environment. 2004. *Groundwater Under the Direct Influence of Surface Water (GUDI) Assessment Guideline.*

USEPA. 1991. *Guidance Manual for Compliance with the Filtration and Disinfection Requirements for Public Water Systems using Surface Water Sources.*

USEPA. 1992. *Consensus Method for Determining Groundwater Under the Direct Influence of Surface Water Using Microscopic Particulate Analysis (MPA).*

APPENDIX F

TEST TIME CALCULATION FOR LOW-PRESSURE AIR TESTING OF INSTALLED PVC SEWER PIPE

(Reproduced from Uni-Bell PVC
Pipe Association Standard UNI-B-6-90,
Recommended Practice for Low-pressure
Air Testing of Installed Sewer Pipe - May 1990)

1.1 TEST TIME CRITERIA

The Ramseier test time criteria requires that no test section shall be accepted if it loses more than Q cubic feet per minute per square foot of internal pipe surface area for any portion containing less than 625 square feet internal pipe surface area. The total leakage from any test section shall not exceed 625 Q cubic feet per minute.

1.2 ALLOWABLE AIR LOSS RATE

A Q value of 0.0015 cubic feet per minute per square foot shall be utilized to assure the Owner of quality pipe materials, good workmanship and tight joints.

1.3 TEST TIME CALCULATION

All test times shall be calculated using Ramseier's equation:

$$T = 0.085 \frac{DK}{Q}$$

Where: T = Shortest time, in seconds, allowed for the air pressure to drop 1.0 psig,

K = 0.000419 DL, but not less than 1.0,

Q = 0.0015 cubic feet/minute/square feet of internal surface,

D = Nominal pipe diameter in inches, and

L = Length of pipe being tested in feet.

For more efficient testing of long test sections and/or sections of larger diameter pipes, a timed pressure drop of 0.5 psig may be used in lieu of the 1.0 psig timed pressure drop. If a 0.5 psig pressure drop is used, the appropriate required test times shall be exactly half as long as those obtained using Ramseier's equation for T cited above.

1.4 SPECIFIED TIME TABLES

To facilitate the proper use of this recommended practice for air testing, the following tables are provided. Table I contains the specified minimum times required for a 1.0 psig pressure drop from a starting pressure of at least 3.5 psig greater than the average back pressure of any groundwater above the pipe's invert. Table II contains specified minimum times required for a 0.5 psig pressure drop from a starting pressure of at least 3.5 psig greater than the average back pressure of any groundwater above the pipe's invert.

TABLE I

MINIMUM SPECIFIED TIME REQUIRED FOR A 1.0 PSIG PRESSURE DROP FOR SIZE AND LENGTH OF PIPE INDICATED FOR Q = 0.0015

1 Pipe Diameter (in.)	2 Minimum Time (min: sec)	3 Length for Minimum Time (ft)	4 Time for Longer Length (sec)	Specification Time for Length (L) Shown (min:sec)							
				100 ft	150 ft	200 ft	250 ft	300 ft	350 ft	400 ft	450 ft
4	3:46	597	.380 L	3:46	3:46	3:46	3:46	3:46	3:46	3:46	3:46
6	5:40	398	.854 L	5.40	5.40	5.40	5.40	5.40	5:40	5:42	6:24
8	7:34	298	1.520 L	7:34	7:34	7:34	7:34	7:36	8:52	10:08	11:24
10	9:26	239	2.374 L	9:26	9:26	9:26	9:53	11:52	13:51	15:49	17:48
12	11:20	199	3.418 L	11:20	11:20	11:24	14:15	17:05	19:56	22:47	25:38
15	14:10	159	5.342 L	14:10	14:10	17:48	22:15	26:42	31:09	35:36	40:04
18	17:00	133	7.692 L	17:00	19:13	25:38	32:03	38:27	44:52	51:16	57:41
21	19:50	114	10.470 L	19:50	26:10	34:54	43:37	52:21	61:00	69:48	78:31
24	22:40	99	13.674 L	22:47	34:11	45:34	56:58	68:22	79:46	91:10	102:33
27	25:30	88	17.306 L	28.51	43:16	57:41	72:07	86:32	100:57	115:22	129:48
30	28:20	80	21.366 L	35:37	53:25	71:13	89:02	106:50	124:38	142:26	160:15
33	31:10	72	25.852 L	43:05	64:38	86:10	107:43	129:16	150:43	172:21	193:53
36	34:00	66	30.768 L	51:17	76:55	102:34	128:12	153:50	179:29	205:07	230:46

TABLE II

MINIMUM SPECIFIED TIME REQUIRED FOR A 0.5 PSIG PRESSURE DROP FOR SIZE AND LENGTH OF PIPE INDICATED FOR Q = 0.0015

1 Pipe Diameter (in.)	2 Minimum Time (min: sec)	3 Length for Minimum Time (ft)	4 Time for Longer Length (sec)	Specification Time for Length (L) Shown (min:sec)							
				100 ft	150 ft	200 ft	250 ft	300 ft	350 ft	400 ft	450 ft
4	1:53	597	.190 L	1:53	1:53	1:53	1:53	1:53	1:53	1:53	1:53
6	2:50	398	.427 L	2:50	2:50	2:50	2:50	2:50	2:50	2:51	3:12
8	3:47	298	.760 L	3:47	3:47	3:47	3:47	3:48	4:26	5:04	5:42
10	4:43	239	1.187 L	4:43	4:43	4:43	4:57	5:56	6:55	7:54	8:54
12	5:40	199	1.709 L	5:40	5:40	5:42	7:08	8:33	9:58	11:24	12:50
15	7:05	159	2.671 L	7:05	7:05	8:54	11:08	13:21	15:35	17:48	20:02
18	8:30	133	3.846 L	8:30	9:37	12:49	16:01	19:14	22:26	25:38	28:51
21	9:55	114	5.235 L	9:55	13:05	17:27	21:49	26:11	30:32	34:54	39:16
24	11:20	99	6.837 L	11:24	17:57	22:48	28:30	34:11	39:53	45:35	51:17
27	12:45	88	8.653 L	14:25	21:38	28:51	36:04	43:16	50:30	57:42	46:54
30	14:10	80	10.683 L	17:48	26:43	35:37	44:31	53:25	62:19	71:13	80:07
33	15:35	72	12.926 L	21:33	32:19	43:56	53:52	64:38	75:24	86:10	96:57
36	17:00	66	15.384 L	25:39	38:28	51:17	64:06	76:55	89:44	102:34	115:23

APPENDIX G

Security Self-Assessment Guide For Owners and Operators of Alberta Drinking Water Systems

Security Self-Assessment Guide For Owners and Operators of Alberta Drinking Water Systems

Introduction

Water systems are critical to every community. Protection of public water systems should be a high priority for local officials and water systems owners and operators to ensure an uninterrupted water supply, which is essential for public health and safety.

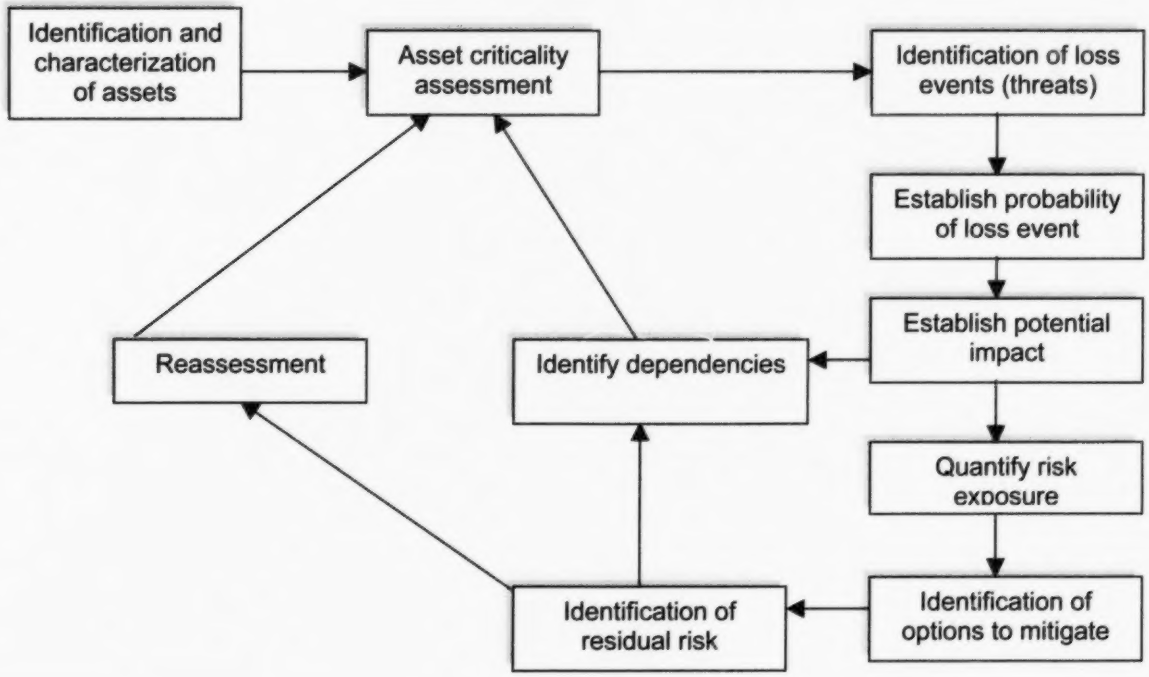
Adequate security measures will help to prevent loss through criminal behaviour including acts of terrorism and every-day vandalism. Proper planning will not only help to prevent such incidents but will mitigate their impact and assist in rapid recovery.

This Security Self assessment guide is designed to help water systems determine possible vulnerable components and identify security measures that should be considered in order to protect the system and the customers it serves.

How to use this self-assessment guide

This document is designed for use by water system personnel when performing vulnerability assessments. Assessments should include, but not be limited to a review of physical facilities, distribution systems, treatment facilities, physical barriers, water collection, water storage, computer networks, chemical storage and handling procedures and operational methods. In other words the scope of this vulnerability assessment should be to review the security of the total water system from source to tap.

The self-assessment has a number of steps as shown on this process overview:


Identification and characterization of assets
Asset criticality assessment
Identification of loss events (threats)
Establish probability of loss event
Establish potential impact
Identify dependencies
Reassessment
Quantify risk exposure
Identification of options to mitigate
Identification of residual risk

a) Identification and characterization of assets

A complete list of all the assets of the total water system should be drafted together with description of these assets and their contribution to the water system. (Addendum 2)

b) Assets Criticality assessment

All critical assets or water system components should be identified. A critical asset is an asset that, if destroyed or lost would prevent the water system from delivering safe clean water in sufficient quantity or at sufficient pressure.

Additional factors which should be considered when assessing criticality of an assets or components are:

- Interdependency with critical infrastructure or services, e.g. hospitals, power facilities, major customers
- Single points of failure
- Availability of redundant back ups
- Replacement time of components or assets
- Potential for loss of public confidence.

c) Identification of loss events (Threats)

This could be as simple as a list of potential threats that a water system might face, or it could be specific to a particular system or geographic location. Common threats to be considered are:

- Vandalism
- Unlawful trespass
- Computer hacking
- Disgruntled employee
- Criminal contamination
- Accidental contamination
- Labour dispute.

d) Establish Probability of loss event

The probability that a threat could occur should be quantified. Individual water systems will decide how probability should be quantified however a rating of High, Medium or Low probability could be used. When assessing probability a number of factors should be considered;

- Relevant history of similar events
- The crime rate in each area

- Major changes in socio-economic or demographic situation
- Intelligence of threats from other agencies.

It is recognized that although some objective input on probability may be received from sources such as Alberta Environment, the Police or RCMP, assessment of probability remains largely subjective. The following table could be used as a means to express probability.

1 **Unlikely**	2 **Moderate**	3 **Likely**	4 **Almost Certain**
3% to 15%	15% to 50%	50% to 70%	> 70%
Small likelihood but could well happen	Less than 50-50 chance	More than 50-50 chance	Almost certain that it will happen, very frequent occurrence
Once in 50 years	Once in 20 years	Once in 10 years	Once per year or more
Low level of threat	Medium Level of threat	High Level of threat	Imminent threat of attack

e) Establish potential impact

The impact of each loss event should be considered in detail and not just in terms of the ability to affect water quality. For example introduction of a small amount of a chemical contaminant into a treat water storage reservoir would have little or no impact on water quality; however this could have serious impact upon public confidence in the water system.

When considering impact, considerable time must be spent on identification of dependant infrastructure and critical customers. If we were to decide that that a loss of supply to < 500 customers was a moderate impact, but one of those customers was the only hospital in a region, then the impact of that loss event should be considered to have a much higher impact, especially if sustained for a long period of time.

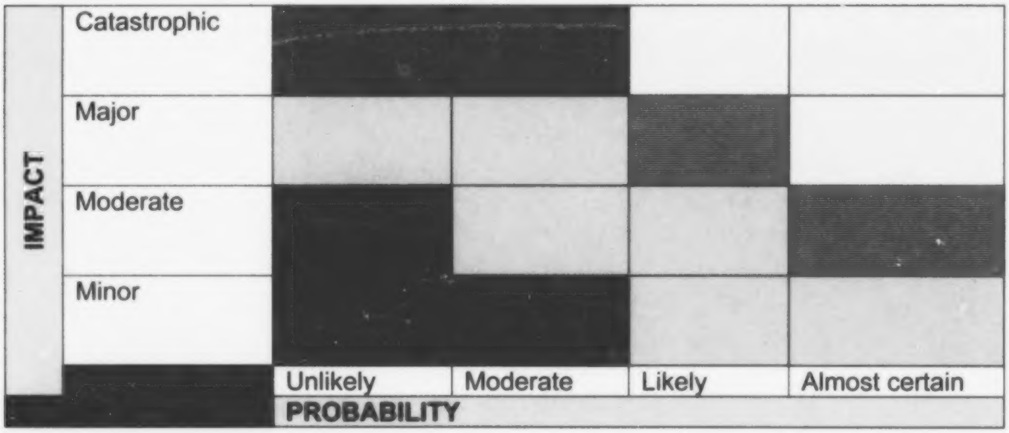
The following table could be used to assist when assessing potential impact. System operators will need to develop their own descriptions and thresholds for what they consider to be Minor, Moderate Major and Catastrophic events.

	1 Minor	2 Moderate	3 Major	4 Catastrophic
Health and Safety of Employees and Public				
Total Financial Loss				
Loss of treatment				
Loss of Pressure				
Loss of supply				
Environmental Impact				
Reputation Impact				
Public Confidence				
Political Impact				
Loss of control ability				

f) Quantify Risk Exposure

Risk can be considered to be the product of probability x Impact. By using the table below we can assess the risk exposure for each identified threat.

For Example: The incidence of Graffiti is very high in some areas. If we considered this as a threat or loss event then we would follow the process above which would show that the probability is almost certain, as our crews have to clean this up on a regular basis. When we consider impact we assess that the impact is minor is all of the assessed areas. When we assess the risk using the matrix we find that this is a Medium level of risk that should be managed by introduction of mitigations and procedural security measures.



IMPACT	Catastrophic	■	■		
	Major	░	░	▩	
	Moderate	■	░	░	▩
	Minor	■	■	░	░
		Unlikely	Moderate	Likely	Almost certain
		PROBABILITY			

■	**Indicates a Low level of risk** - minimal security measures necessary. Risk should managed through the introduction of mitigation strategies or changes in procedural security, or accepted as a low business risk.
▒	**Indicates a Medium Level of risk** – Risk should be managed by the introduction of mitigation strategies and high levels of procedural security.
■	**Indicates a High Level of risk** – Maximum-security measures should be in place, providing layers of Deterrence, Delay, High probability of detection and rapid effective response. Insurance coverage essential but may not be able to provide adequate coverage to prevent significant liability. Due diligence is required, including utilisation of appropriate expertise and validation of assessed data.
□	**Indicates a severe level of risk**.- Maximum security measures should be in place, together with recovery plans, mutual aid agreements and availability of critical spares.

g) Identification of countermeasures, their costs and trade-offs.

   i. Identify potential security countermeasures to reduce the vulnerabilities.

   ii. Estimate the cost of the countermeasures.

   iii. Conduct a cost-benefit and trade-off analysis.

   iv. Prioritize options and recommendations for senior management.

The results of the Threat and Risk analysis can then be plotted and recorded on the matrix shown at Addendum 3. This form can also be used to show recommended mitigations and ongoing mitigation strategies.

Self-Assessment Worksheet

A self–assessment work sheet is shown at Addendum 4. This guideline will assist operators to ask the right questions when reviewing the security regimes in place at various plants and facilities. The guideline is designed to prompt operators to look at various security aspects and determine a course of action appropriate to their operation. This guideline is not intended to be a security manual or to deliver advice or instruction to operators on the exact security measures to be installed at each facility. Operators should seek specialized security advice from qualified practitioners if they have no internal resources assigned to the security function.

Upon completion of this self-assessment system owners/ operators should complete the certificate shown at Addendum 1 and forward this to Alberta Environment.

ADDENDUM 1

Certification of Completion of Security Self Assessment
Drinking Water System

To: Alberta Environment

Name of Water System:

Administrative Address of Water System:

I certify that on -----(Date)----- a Security Self Assessment was performed on the above-indicated Drinking Water system. I further certify that the information in this self-assessment has been completed to the best of knowledge and that the appropriate parties have been notified of the assessment and the recommended steps to be taken to enhance the security of the water system. Furthermore a copy of the completed self-assessment will be retained at the administrative offices, in a secure location, and will be available for inspection by Alberta Environment as requested.

I further certify that --------(Name)--------- has been delegated with the responsibility for security as it pertains to this water system.

Yours sincerely

______________________________________Signed

______________________DATE

ADDENDUM 2

Component type	Number & Location	Description	Criticality or single point of failure	Comments
Source Water				
Ground Water				
Surface Water				
Other				
Treatment Plant				
Buildings				
Pumps				
Process controls				
Treatment Chemical storage				
Laboratory chemicals and storage				
Other				
Storage				
Reservoirs				
Storage Tanks				
Pressure Tanks				
Other				
Power supply				
Primary Power supply				
Auxiliary Power supply				
Other				
Distribution system				
Pumps				
Pressure Reducing Valves				

Component type	Number & Location	Description	Criticality or single point of failure	Comments
Pressure sustaining valves				
Other Valves				
Hydrants				
Back flow preventers				
Other				
Communications & Control				
SCADA Assets				
- RTU's				
- PLC's				
- IED's				
- Control terminals				
- Servers				
- Switches / firewalls etc.				
Telephony systems				
Cell Phone systems				
Radio systems				
Business network systems				
Other				
Administration				
Office buildings				
Computer systems				
File storage				
Service vehicles				
personnel				
Other				

Component type	Number & Location	Description	Criticality or single point of failure	Comments
Critical infrastructure customers				
Power Plants				
Hospitals				
Schools				
Food processing plants				
Major industry				
Other				

ADDENDUM 3

Threat and Risk Assessment

ASSET	**Water Treatment Plant**					**CRITICALITY**	**HIGH**
Threat or hazard	**Vulnerability**	**Prob.**	**Impact**	**Risk**	**Recommended mitigation(s)**	**Residual Risk**	**On going mitigation**
Vandalism (EXAMPLE)	Facilities are not fenced	Moderate	Moderate	MED	Consider installation fencing Install signage	LOW	▪ Staff awareness and vigilance ▪ Maintenance program for fences

ADDENDUM 4

Drinking Water systems **Security Self-Assessment worksheet**	
Name of Water System	
Number of customers served	
Administrative address	
Security Manager Name (Or manager with Security Responsibility)	
Telephone Number	
Date review completed	
Persons involved in review process	
Date of review of outstanding action items	

Question	**Answer**	**Action Needed / Taken**
Personnel		
Have you appointed a security manager or senior manager with responsibility for security of the water system?		
When hiring personnel do you perform background checks or require criminal record checks?		
Do you have a security awareness training program?		

ADDENDUM 4

Drinking Water systems Security Self-Assessment worksheet		
When terminating employees do you have a defined 'End of Service' process, which disables logical and physical access and requires the return of keys, ID cards and other security related items?		
Have water system personnel been advised to report security vulnerability concerns or suspicious behaviour to the Security Manager or department?		
Emergency Reaction Plans		
Do you have a written Emergency Reaction Plan (ERP) which conforms to Canadian Standards Association standard CSA z731?		
IS there someone designated with the responsibility to ensure the ERP is updated and reviewed?		
Has the ERP been tested in the last year?		
Are records kept of all ERP tests? When was the ERP last reviewed and updated?		
Are the contact lists shown in the ERP up to date with correct information?		
Have first responders (FIRE, EMS) been made aware of the plan and contributed to its formation or review?		
Do you have a Business Continuity Plan (BCP)?		

ADDENDUM 4

Drinking Water systems **Security Self-Assessment worksheet**		
When was the BCP last tested, reviewed and updated?		
Who is responsible for the maintenance of the BCP?		
Physical Security		
Is access to critical components of the water systems restricted to authorized personnel only?		
How is access control achieved?		
Are employees and authorized contractors required to carry or display ID badges?		
Do you use non-reproducible keys? If so how is distribution of these keys achieved? Do you perform regular audits of keys and require that keys be signed in and out?		
Are buildings locked when not in use? Is there a process or control in place to ensure buildings are locked? Are service vehicles locked when not in use? Is critical equipment stored in vehicles?		

ADDENDUM 4

Drinking Water systems **Security Self-Assessment worksheet**		
Do you use an electronic card access system? Are different levels of access granted to employees and contractors based on the principle of the least amount of access necessary to perform their role? Are access levels routinely audited to ensure currency? What procedures are in place to deal with employees who transfer to other responsibilities?		
Have buildings been designed to incorporate Crime Prevention Through Environmental Design (CPTED) principles? Are new buildings required to incorporate CPTED principles? Are new building plans subject to approval from the security manager or architect with CPTED experience?		
Are critical buildings fenced? Are fences at least 2130 mm high with 3 strands of barbed wire on top to prevent climb over?		

ADDENDUM 4

Drinking Water systems **Security Self-Assessment worksheet**		
Are fences subject to regular inspection and maintenance? Is a clear zone of 3 metres in place on either side of all fence lines? Are gates properly secured with high security locks, electronic access control or high security padlocks? Are adequate no Trespass signage in place beside the main entrance and at all four corners of each facility, as required by the Trespass to Premises Act? Is this signage in good condition and legible? Are there adequate warning signs in place to warn of site dangers? High Voltage, deep water, Chemicals etc.		
Are intrusion detection systems installed? Where and how are these monitored? What is the response to an alarm activation?		
Are reservoir and tank access hatches and vents locked? Are tamper proof hinges and locks used on Reservoir and tank access hatches?		

Drinking Water systems Security Self-Assessment worksheet		
Are these regularly inspected? Are any intrusion detection systems installed on reservoir or tank access hatches? Is there a response plan for alarm activations involving reservoir and tank access hatches? Is the area around critical components free of objects which may be used for breaking and entering? (e.g. Large rocks, cement blocks, pieces of wood, ladders, valve keys and other tools)		
Do you have a neighbourhood watch program for you water system?		
Do wellheads, pumps and treatment facilities have back up power supplies? If so, when were these last tested? Is there a preventative maintenance program in place for back up generators? Is there spill containment in place for diesel fuel tanks required by back-up generators?		
Does secured manhole covers or secondary locking devices protect critical valves? Are these inspected on a regular basis?		

ADDENDUM 4

Drinking Water systems
Security Self-Assessment worksheet

SCADA Security		
Does access to SCADA systems require a user name and a strong password? Are passwords changed on a regular basis? Are controls in place to stop the sharing of passwords? Are separate accounts created for contractor access to SCADA? Are user accounts removed from SCADA upon termination of employees or contracts? Is there a single point of accountability for SCADA systems?		
Have all SCADA connections with the Internet or modems been identified and documented. Are these connections protected by Firewalls or other form of connection that authenticates user sessions.		
Have redundant or obsolete connections been removed or deactivated?		
Are SCADA operating systems properly patched?		

ADDENDUM 4

Drinking Water systems Security Self-Assessment worksheet		
Is there a process in place to ensure prompt patching of identified vulnerabilities? Are patches tested prior to deployment?		
Are SCADA applications properly patched and upgraded with the latest version? Is there a process in place to ensure SCADA applications are promptly patched and to ensure currency of the latest version?		
IS the SCADA System connected to a business or administration network? Is the SCADA system properly partitioned from the business network?		
Do you have a disaster recovery plan for SCADA systems? Has this plan been tested and reviewed?		
Are details of you water system posted on an Internet site? Have these details been reviewed to ensure that no information is displayed which may be used to compromise the water system?		